US Army Corps of Engineers ®
Portland District

Mount St. Helens Project
Cowlitz River Levee Systems

2009 Level of Flood Protection Update Summary

Cowlitz River at Longview/Kelso, Washington

February 4, 2010

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Executive Summary

USACE periodically updates the Levels of Protection (LOP) for the Cowlitz River levees as part of on-going activities of the Mount St. Helens project. The most recent comprehensive LOP update prior to 2009 was performed in 1997 for the Cowlitz River Flood Hazard Study. Since 1997, there have been several interim assessments of the levee system, however a complete study of the hydrology, hydraulics, and levee index points were not performed for these interim assessments. The current (2009) LOP assessment for the Cowlitz River levees incorporates a detailed review of each component of the 1997 Cowlitz River Flood Hazard Study. This update includes:

- Revisions to the Fragility Curve for the levees;
- Revisions to the Cowlitz River discharge-frequency curve;
- Revisions to the stage-discharge relationships based on a comprehensive Cowlitz River survey, performed in August 2009 by David Evans and Associates, from the Toutle River confluence, approximately River Mile (RM) 20, to the Columbia River confluence (RM 0);
- Revisions to hydraulic and hydrologic uncertainty;

A complete investigation of Safe Water Levels (SWL) is performed for the four levees along the lower 20 miles of the Cowlitz River. Index points are identified along the levee at locations with the least amount of flood protection. Locations of the index points changed in many cases from those used in the previous LOP estimates. The SWL's are used to develop geotechnical fragility curves for use in Level of Protection (LOP) evaluations at selected index points. The fragility curves are developed assuming increasing probability of failure from zero at the SWL to 1.0 at the top of the levee. In most cases a straight line is assumed between the probability of failure at the SWL and the top of the levee. An exception was made at the Lexington levee where a 50 percent probability of failure is assumed when the water surface is at the top of the levee and a 100 percent chance of failure is assumed when the water surface is above the top of the levee. Additionally, for cases where the SWL is determined to be the same elevation as the top of the levee, 100 percent probability of failure is assumed when the water surface is above the top of the levee and zero percent probability of failure is assumed when the water surface is below the top of the levee.

The 2009 update to the flood frequency curves incorporate a more robust hydrologic methodology and include a longer period of recorded data. In the updated hydrology, the actual and estimated regulated discharges for the period of record at Castle Rock (1927 to 2009) are used to create the regulated discharge-frequency curve. Measured regulated peak discharge data is used from the years of significant regulation (1969-2009). For years prior to regulation, a combination of detailed routings for larger events and a regulated-unregulated relationship for events less than 90,000 cfs are used to estimate regulated peak data. Synthetic hydrology is developed to supplement the 83-years of observed data and add resolution to the 0.01 to the 0.001 annual exceedence probability (AEP) range. The results are merged into a composite discharge-frequency relationship representing the mean frequency for a given discharge.

The new regulated discharge-frequency curve at Castle Rock is graphically drawn, first through the 83 plotted points and then using the results from the synthetic analysis for the upper and lower frequencies. Uncertainty for the updated hydrology at Castle Rock is related to the length of the systematic record. An equivalent years-of-record (EYR) of 90 years is calculated based on 83 years of data adjusted for the historical period and for additional uncertainty from estimating regulated peaks for pre-regulation years.

Stage discharge curves for each index point are developed from a 1-D, fixed bed, steady state, HEC-RAS model that was calibrated to a January 2009 flood event with a 40 year frequency. Geometry for the

hydraulic model is based on a hydrosurvey performed in August 2009, and represents the most current geometric characteristics relative to the calibration event. Uncertainty in the stage-discharge curves is computed through the analysis of three components; natural uncertainty, model uncertainty, and sediment uncertainty. All three components were combined to compute the total uncertainty for the stage-discharge curves.

Utilizing the updated levee fragility curves, the updated flood frequency curves, and updated stage-discharge relationships; HEC-FDA is used to evaluate the probability of non-exceedance for each of the levees associated with the Cowlitz River. Level of protection is evaluated at each levee index point by linear interpolating the flood frequency corresponding to the 90 percent probability of non exceedance from the HEC-FDA results. Each levee contains multiple index points, the final reported LOP for each levee is the lowest calculated LOP for all of the index points along the levee. The results from this analysis are included in the following table.

Index Point		River Mile (RM)	Safe Water Level (ft)	Top of Levee (ft)	Authorized LOP (1/years)	Discharge at LOP (cfs)	Expected Stage at Authorized LOP (ft)	Current LOP (1/years)	Probability of Containing the 0.4% (1/250-yr) Annual Chance Exceed. Flood	Probability of Containing the 1.0% (1/100-yr) Annual Chance Exceed. Flood
Castle Rock 1	CRIP 1	17.42	65.8	65.8	118	115,000	58.2	468	99.7%	100.0%
Castle Rock 2	CRIP 2	17.00	57.3	60.9	118	115,000	56.2	109	68.3%	93.5%
Castle Rock 3	CRIP 3	15.91	58.5	58.5	118	118,500	54.2	160	84.6%	99.1%
Lexington 1	LXIP 1	8.64	38.2	45.7	167	126,100	37.8	202	88.5%	97.8%
Lexington 2	LXIP 2	8.30	42.6	42.6	167	126,100	35.7	326	98.8%	100.0%
Kelso 1	KLIP 1	7.00	37.7	37.7	143	122,400	30.6	>500	99.7%	100.0%
Kelso 2	KLIP 2	6.19	37.4	40.3	143	122,400	29.4	>500	100.0%	100.0%
Keslo 3	KLIP 3	4.02	33.5	34.5	143	122,400	26.3	>500	100.0%	100.0%
Keslo 4	KLIP 4	3.70	30.4	33.4	143	122,400	25.9	470	99.4%	100.0%
Longview 1	LVIP 1	4.90	35.1	35.1	167	126,100	26.8	>500	100.0%	100.0%
Longview 2	LVIP 2	4.68	34.8	37.4	167	126,100	27.0	>500	100.0%	100.0%
Longview 3	LVIP 3	3.59	32.8	32.8	167	126,100	25.7	>500	100.0%	100.0%
Longview 4	LVIP 4	3.27	32.0	32.5	167	126,100	24.9	>500	100.0%	100.0%

Note: All Elevations given in ft NAVD88

COMPLETION OF INDEPENDENT TECHNICAL REVIEW

The HEC staff has completed the review of the *Mount St. Helens Project Cowlitz River Levee Systems - 2009 Level of Flood Protection Update* report. Notice is hereby given that an independent technical review, that is appropriate to the level of risk and complexity inherent in the project, has been conducted as defined in the Quality Control Plan. During the independent technical review, compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This included review of: assumptions; methods, procedures, and material used in analyses; the appropriateness of data used and level obtained; and reasonableness of the result, including whether the product meets the customer's needs consistent with law and existing Corps policy. The independent technical review of the performance analyses was accomplished by Beth Faber, PhD, PE, Woodrow Fields and Michael Deering, PE, D.WRE. All comments resulting from ITR have been resolved.

Technical Review Team Leader — 1/20/10 Date

Project Manager — 1/29/10 Date

CERTIFICATION OF INDEPENDENT TECHNICAL REVIEW

As noted above, all concerns resulting from independent technical review of the project have been fully resolved.

Donald R. Chambers
Chief, Engineering and Construction Division — 2/1/10 Date

EC-T
Rev 1 Feb 08

COMPLETION OF INDEPENDENT TECHNICAL REVIEW

The District has completed a discharge-frequency analysis of the lower 20 miles of the Cowlitz River in Washington State. The complete product reviewed is included as Appendix B, *Discharge –Frequency Analysis,* in the *Mount St. Helens Project Cowlitz River Levee Systems 2009 Level of Flood Protection Update Summary* report. Notice is hereby given that an independent technical review, that is appropriate to the level of risk and complexity inherent in the project, has been conducted as defined in the Quality Control Plan. During the independent technical review, compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This included review of: assumptions; methods, procedures, and material used in analyses; alternatives evaluated; the appropriateness of data used and level obtained; and reasonableness of the result, including whether the product meets the customer's needs consistent with law and existing Corps policy. The independent technical review was accomplished by Beth Faber, PhD, PE; Hydrologic Engineering Center; Davis, CA. All comments resulting from ITR have been resolved.

Technical Review Team Leader	1/20/10 Date
Project Manager	1/29/10 Date

CERTIFICATION OF INDEPENDENT TECHNICAL REVIEW

As noted above, all concerns resulting from independent technical review of the project have been fully resolved.

Donald R. Chambers Chief, Engineering and Construction Division	2/1/10 Date

EC-T
Rev 1 Feb 08

COMPLETION OF AGENCY TECHNICAL REVIEW

NWW District has completed the agency technical review of the Cowlitz River Levees Safe Water Level Study Report. Notice is hereby given that an agency technical review, that is appropriate to the level of risk and complexity inherent in the project, has been conducted as defined in the Quality Control Plan. During the agency technical review, compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This included review of: assumptions; methods, procedures, and material used in analyses; alternatives evaluated; the appropriateness of data used and level obtained; and reasonableness of the result, including whether the product meets the customer's needs consistent with law and existing Corps policy. The agency technical review was accomplished by Walla Walla District. All comments resulting from ATR have been resolved.

Technical Review Team Leader — 11 Jan 2010 Date

Project Manager — 1/25/10 Date

CERTIFICATION OF AGENCY TECHNICAL REVIEW

Significant concerns and the explanation of the resolution are as follows:

(If any, describe the major technical concerns, possible impact, and resolution)

As noted above, all concerns resulting from agency technical review of the project have been fully resolved.

Chief, Engineering and Construction Division — 2/1/10 Date

EC-T
Rev 1 Feb 08

Abbreviations and Acronyms

cfs	cubic feet per second
AEP	Annual Exceedance Probability
CNP	Non-exceedance Probability
FDA	Flood Damage Reduction Analysis
HEC	Hydrologic Engineering Center
LiDAR	Light Detection and Ranging
LOP	level of flood protection
NAVD	National American Vertical Datum
POR	Period of Record
PNP	probability of non-failure
RAS	River Analysis System
RM	river mile(s)
SRS	sediment retention structure
SWL	safe water level
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey

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1. INTRODUCTION

1.1. OVERVIEW

USACE periodically updates the Levels of Protection (LOP) for the Cowlitz River levees as part of on-going activities of the Mount St. Helens project. The most recent comprehensive LOP update prior to 2009 was performed in 1997 for the Cowlitz River Flood Hazard Study. Since 1997, there have been several interim assessments of the levee system, however a complete study of the hydrology, hydraulics, and levee index points was not performed for these interim assessments. The current (2009) LOP assessment for the Cowlitz River levees incorporates a detailed review of each component of the 1997 Cowlitz River Flood Hazard Study. This update includes:

- Revisions to the stage discharge relationships resulting from a comprehensive Cowlitz River hydrosurvey, performed in August 2009 by David Evans and Associates, from the Toutle River confluence at approximately RM 20 to the Columbia River confluence;
- Revisions to the Cowlitz River discharge frequency curve;
- Revisions to hydraulic and hydrologic uncertainty;
- Revisions to the fragility curves for the levees.

1.2. AUTHORIZED LEVELS OF FLOOD PROTECTION

The U.S. Army Corps of Engineers (USACE) was directed by Congress to maintain an authorized level of flood protection (LOP) in four communities along the Cowlitz River that is not less than described in the *Mount St. Helens, Washington, Decision Document, Toutle, Cowlitz and Columbia Rivers* (USACE 1985). As shown in Figure 1-1, the Cowlitz River levee reaches include the Castle Rock levee [River Mile (RM) 15.91 to 17.66], Lexington levee (RM 7.12 to 9.53), Kelso levee (RM 1.59 to 7.3), and the Longview levee (RM 1.59 to 5.57).

In the 1980s and early 1990s, the levels of protection for Castle Rock, Lexington, Longview, and Kelso were determined using a deterministic approach in which median values of flood stages were compared to levee safe water levels (SWL). The SWL was evaluated as the highest flood level for which reasonable assurance could be made that the levee would not fail, and was restricted to no less than 3 ft below the levee top in order to provide freeboard for uncertainties. The SWL was often dictated by encroachments to the levees. The level of protection was evaluated as the highest average-return-period-event whose median-value flood profile was no higher than the SWL at all points along the levee[11].

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[1] Historical context of the Level of Protection from 1985 is based on Corps of Engineers personnel who was personally involved with studies that led to the original LOP estimate for the levees along the Cowlitz.

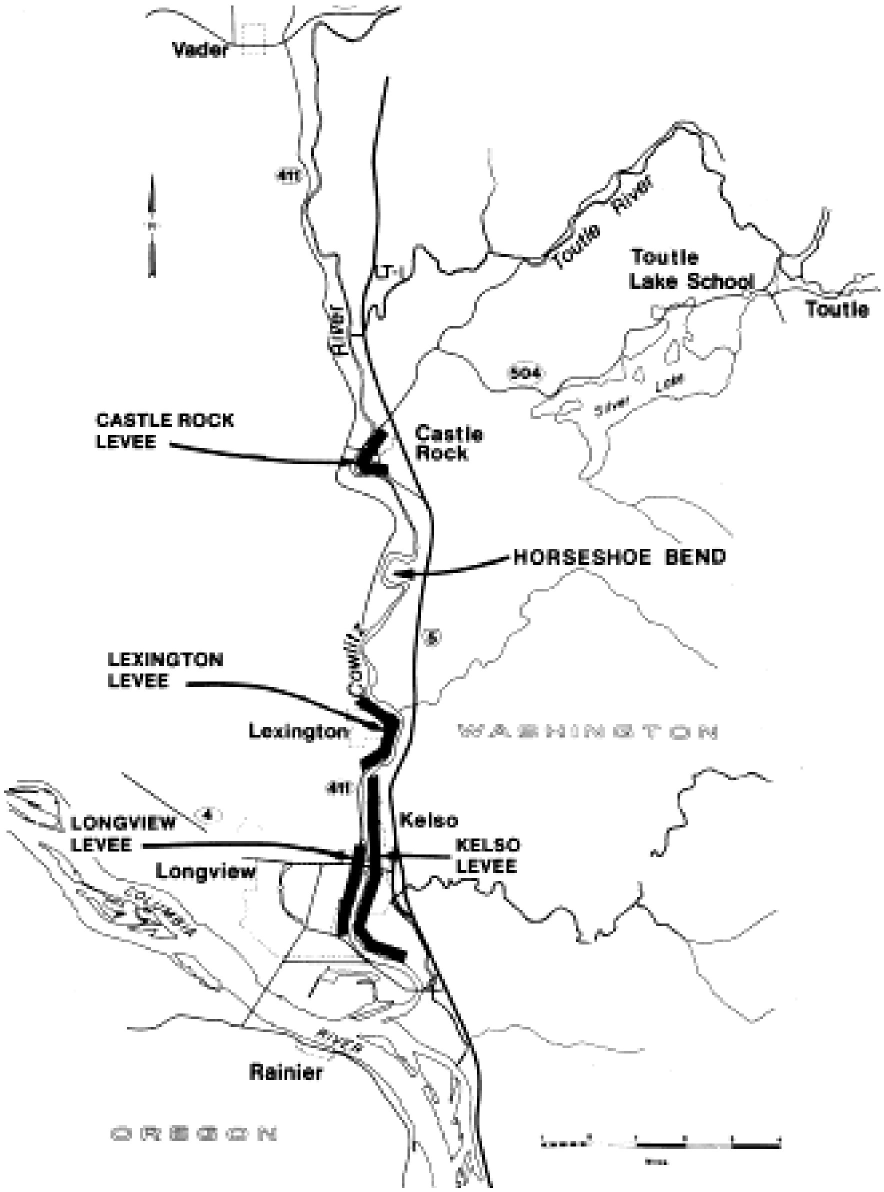

Figure 1-1. Map showing locations of individual Cowlitz River levee reaches that are part of the Mount St. Helens project.

Congressional direction provides that the USACE must maintain the LOP for the Cowlitz River levee systems (Figure 1-1) through the end of the project planning period, which is 2035. The authorized levels of flood protection are expressed as recurrence interval floods that result in the levee system capacity exceedance or failure. For the Cowlitz River levee systems, the authorized protection levels are shown in Table 1-1.

Table 1-1. Authorized LOP for the Cowlitz Levee projects given in terms of Annual Exceedance Probability Return Period in Years

Levee Location	Levee Length (miles)	Authorized LOP
Kelso	5.7	1/143
Longview	2.4	1/167
Lexington	2.7	1/167
Castle Rock	1.5	1/118

1.3. DEFINITION OF LEVEL OF FLOOD PROTECTION

The LOP assessment for the Cowlitz River levee projects is defined as the recurrence interval of the flood event that results in the levee protection system capacity exceedance or failure (USACE, ETL 1110-2-570, draft 2007). The recurrence interval where failure is evaluated is based on the authorized levels published in USACE 1985 and summarized in Table 1-1. For the given hydrometeorological event shown in Table 1-1, failure can be assessed from a Conditional Non-exceedance Probability (CNP) that represents a likelihood that a specific target will not be exceeded (USACE, EM 1110-2-1619). Borrowing from guidance on levee certification, the target used in the level of protection evaluation for the Cowlitz River levees is determined such that there is 90 percent assurance of providing protection from overtopping by the authorized chance exceedance flood (USACE, ETL 1110-2-570). This criteria assumes that the expected stage at the authorized level of protection is at least three feet below the top of the levee. Overall uncertainty in determining the LOP is included in each component of the evaluation.

1.4. DETERMINING THE LEVEL OF FLOOD PROTECTION

The procedure and methodology for determination of the LOP for the Cowlitz River levees utilizes a probabilistic based analysis approach at designated index locations along each levee system. Index locations are chosen based on a detailed assessment of levee conditions and represent critical locations along the levee that provides the least amount of flood protection. Three key factors were involved in the LOP analysis for the Cowlitz Levee system:

1. ***Geotechnical or levee stability*** as defined by the determination of probability of failure versus stage function (fragility curve). For the level of protection analysis, levee condition is incorporated in the fragility curves at each index location. Index locations within the four Cowlitz River levee project reaches represent portions of the levee system that may be considered as a unit for analysis purposes and have representative properties. A levee reach is the unique entity having different properties than other reaches of the levee system and is used to determine the condition of the levee system. Levee condition and the development of the fragility curve are discussed in more detail in Section 2.
2. ***Hydrologic condition*** as defined by the flow frequency curve for the Cowlitz River. Development of the flow frequency curve and the determination of the hydrologic condition are discussed in detail in 3.
3. ***Hydraulic condition*** as defined by hydraulic uncertainty of the mean or expected value stage-discharge curve developed by computer modeling of the Cowlitz River. Expected value stage-discharge curves are developed at the index locations over a specified range of recurrence interval flood events. Twenty-three recurrence interval flood events were used to develop the stage-discharge curve, ranging from the 99.99% to 0.01% exceedance probability in any given year. Hydraulic condition is discussed in Section 4.

Utilizing the updated levee fragility curves, the updated flood frequency curves, and updated stage-discharge relationships; HEC-FDA is used to evaluate the probability of non-exceedance (or assurance) of containing a given %-change exceedance event for each of the levees associated with the Cowlitz River. No economic analysis is included in the FDA modeling of the Cowlitz Levees. Level of protection is evaluated at each levee index point by linear interpolating the flood frequency corresponding to the 90 percent probability of non exceedance from the HEC-FDA results. Each levee contains multiple index points, the final reported LOP for each levee is the lowest calculated LOP for all of the index points along the levee.

1.5. MONITORING

The USACE periodically updates the Levels of Protection (LOP) for the Cowlitz River levees as part of the ongoing activities of the Mount St. Helens project (Table 1-2). The 2009 update is the most comprehensive update since the 1997 Cowlitz River Flood Hazard Study. Since 1997, however, LOP has been updated periodically. For each update, combinations of new data were used to assess current protection levels. Table 1-2 summarizes all the updates to the LOP that have occurred since 1997 and what new data was incorporated for the corresponding update.

Table 1-2. Cowlitz River level of flood protection updates.

Level of Protection Update	Hydrographic/LiDAR* Survey	Revised Hydrology
September 1997 Cowlitz Flood Hazard Study	June 1996	Yes
2002 Mount St. Helens Engineering Reanalysis	June 1996	No
2004 Level of Protection Update	August 2003 (hydrographic survey RM 0-20)	No
2006 Level of Protection Update	April 2006 (hydrographic survey RM 0-10)	No
2007 Level of Protection Update	December 2006 (hydrographic survey RM 0-10)	No
2009 Level of Protection Update *(Current Update)*	August 2009 (hydrographic survey RM 0-20) 2007 LiDAR	Yes

* Light Detection and Ranging (LiDAR) is a remote sensing system used to collect topographic data.

Cowlitz River channel cross sections are used to monitor changes in the rivers ability to pass a given discharge or channel capacity. Increases or decreases in sediment supply may affect the channel capacity and ultimately its stage-discharge curve. Development of the stage-discharge curves is discussed in Section 4.

1.6. STUDY SCHEMATIC

A complete level of protection analysis for the Castle Rock, Lexington, Kelso, and Longview levees is presented in the following chapters. Level of protection for the Coweeman levee historically has not been included in the level of protection for the Cowlitz River levee system and is not included in this analysis. To support the level of protection analysis a steady state hydraulic model consisting of approximately 100 surveyed cross sections and seven (7) bridge crossings was created and calibrated to 11 calibration points.

Figure 1-2 shows cross section alignments, calibration points, and levee information and will be referred to throughout this document for general spatial reference.

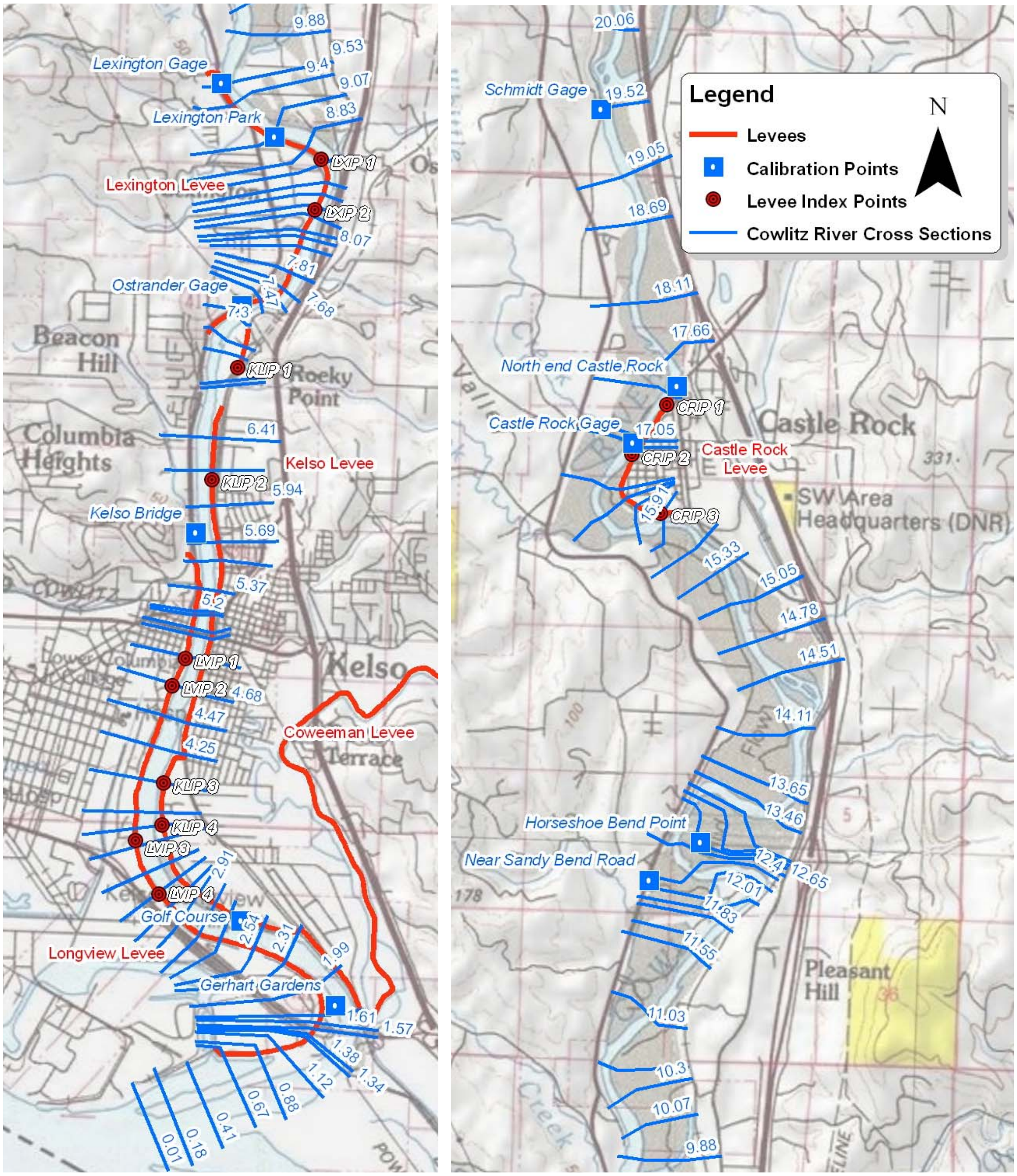

Figure 1-2. Schematic of the Level of Protection Study for the Lower Cowlitz River

2. LEVEE ANALYSIS

2.1. INTRODUCTION

In March, 2009 a Safe Water Level Study was performed for the levee system associated with the Cowlitz River from RM 0 to RM 20. Five (5) levees were assessed in this study; Castle Rock, Lexington, Longview, Kelso, and Coweeman levees; however, only the Castle Rock, Lexington, Longview, and Kelso levees were included in this level of protection update.

The 2009 Safe Water Level Study involved a review of the construction history, inspection history, available reports and drawings, survey data, and multiple site visits for each levee system. Each levee was divided into reaches of similar characteristics. The reaches were then screened to determine the critical points along each levee system, identified as Index Points for the level of protection analysis. The index points were chosen assuming that the diking districts will raise certain known low spots during significant flood events. Three points were identified for the Castle Rock levee, two points for the Lexington Levee, four points were identified for the Kelso Levee, and four points were identified for the Longview Levee. Table 2-1 summarizes the river mile and description of each index point for each levee. Detail description of the Safe Water Level Study is included in Appendix D.

Table 2-1. Cowlitz River Levee System Index Points

Designation	Index Point Description	River Mile
	Castle Rock Levee	
CRIP 1	Approximately 1,500 ft upstream of Castle Rock Bridge	17.42
CRIP 2	Just downstream of Castle Rock Bridge	17.00
CRIP 3	Road crossing by sewage treatment plant	15.91
	Lexington Levee	
LXIP 1	Riverside Park	8.64
LXIP 2	Lexington Across from Mobile Home Park	8.30
	Longview Levee	
LVIP 1	Upstream End of Country Fairgrounds	4.90
LVIP 2	Downstream End of Country Fairgrounds	4.68
LVIP 3	Across from Highway 411	3.59
LVIP 4	Across from Highway 432	3.27
	Kelso	
KLIP 1	Across from Rocky Point	7.00
KLIP 2	End of Pacific Ave	6.19
KLIP 3	Upstream end of Golf Course	4.02
KLIP 4	Upstream end of Golf Course	3.70

2.2. METHODOLOGY

Each identified critical point along the levee system represents an index point where a comparison is made with the Cowlitz River hydraulic model. At each index point the top of levee elevations and safe water elevations were used to describe the geotechnical fragility curve of the levee system. Safe Water Levels (SWLs) represent the highest flood level for which reasonable assurance can be made that the levee will not fail. This level is further defined as the river stage at which only normal surveillance and minor remedial work will be required during normal flood periods and close surveillance during extended periods. For this reason the probability of failure at or below the SWL is zero and increases above the SWL up to the top of the levee where the probability of failure is 100 percent when the levee is overtopped. Figure 2-1 graphically depicts the three different zones that make up the levee fragility curves.

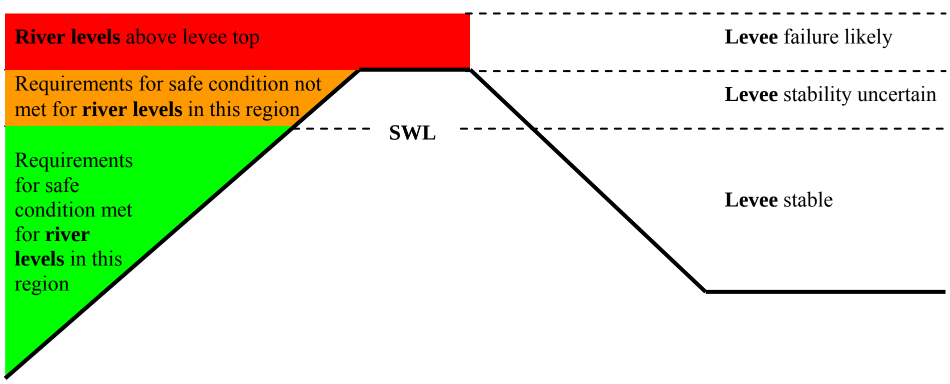

Figure 2-1. Representation of Safe Water Level

Uncertainty of the failure mechanism is inherent in the definition of the failure curve used in the level of protection analysis. Variation of the probability from the SWL to the top of the levee incorporates the geotechnical uncertainty of levee failure. The fragility curves are developed assuming increasing probability of failure from zero at the SWL to 1.0 at the top of the levee. In most cases a straight line is assumed between the probability of failure at the SWL and the top of the levee. An exception was made at the Lexington levee where a 50 percent probability of failure is assumed when the flood stage reaches the top of the levee and a 100 percent chance of failure is assumed when the flood stage overtops the levee.

In some locations, where significant improvements have been made to the levee (such as the cutoff wall installed along the Castle Rock Levee upstream of the Castle Rock Bridge) or where there is significant backfill behind the levee (such as the downstream index point along the Lexington Levee and the second index point from the downstream end of the Longview Levee) the safe water level was defined to be the same as the top of levee. That is, there is zero chance of failure of the levee section as long as the water surface is below the top of the levee. Once the levee is overtopped there is 100 percent chance of failure. Uncertainty at these locations has been reduced to zero due to the significant improvements or stabilizing characteristics associated with the corresponding index point.

Each index point located along the levee system was identified with a different probability of failure curve. The failure curves for each index point are summarized in the following section and described in more detail in Appendix D.

2.3. FRAGILITY CURVES

Probability of failure curves (fragility curves) derived from the Safe Water Level Study for the Cowlitz River levee system are summarized in Table 2-2. To facilitate HEC-FDA interpolation routines, fragility curves presented Table 2-2 were inputted as a line discretized into 10 points. Appendix A includes the HEC-FDA input for the levee uncertainty and the complete Safe Water Level Study is included in Appendix D.

Table 2-2. Fragility Curves for the Cowlitz River Levee System

Index Point	River Mile	Probability of Failure		
		0	0.5	1.0
		(Safe Water Level)		(Top of Levee)
Castle Rock Levee				
CRIP 1	17.42	65.80	65.80	65.80
CRIP 2	17.00	57.30	59.10	60.90
CRIP 3	15.91	58.50	58.50	58.50
Lexington Levee				
LXIP 1	8.64	38.20	45.69	45.70
LXIP 2	8.30	42.60	42.60	42.60
Longview Levee				
LVIP 1	4.90	35.10	35.10	35.10
LVIP 2	4.68	34.80	36.10	37.40
LVIP 3	3.59	32.80	32.80	32.80
LVIP 4	3.27	32.00	32.25	32.50
Kelso Levee				
KLIP 1	7.00	37.70	37.70	37.70
KLIP 2	6.19	37.40	38.85	40.30
KLIP 3	4.02	33.50	34.00	34.50
KLIP 4	3.70	30.40	31.90	33.40

* Safe Water Level and Top of Levee elevations obtained from the Safe Water Level Study included in Appendix D.

3. DISCHARGE-FREQUENCY ANALYSIS

3.1. OVERVIEW

Level of protection (LOP) for the Cowlitz River levees is related to the frequency of discharges experienced in the lower Cowlitz River. This relationship is typically expressed as discharge-frequency curves at key points along the river. Since the Cowlitz River discharges at Castle Rock and below are significantly impacted by flood control and power operations at Mayfield and Mossyrock Dams, regulated discharge-frequency curves are used to assess LOP.

An existing regulated discharge-frequency curve is from a hydrologic analysis presented as part of the 1997 Cowlitz River Flood Hazard Study. In the draft 2008 LOP update, a hydrologic analysis was initiated by Portland District to create a new regulated discharge frequency curve using a more robust methodology and an updated period of record. The approach taken in the 2008 analysis is finalized and presented in its entirety in Appendix B of this report.

3.2. APPROACH OF THE FLOW FREQUENCY ANALYSIS

In the updated hydrology, the actual and estimated regulated discharges for the period of record at Castle Rock (1927 to 2009) are used to create the regulated discharge-frequency curve. Measured regulated peak discharge data is used from the years of significant regulation (1969-2009). For years prior to regulation, a combination of detailed routings for larger events and a regulated-unregulated relationship for events less than 90,000 cfs are used to estimate regulated peak data. Historic data are used to define historic periods that affect the plotting position of measured results and the discharge-frequency statistics. Incorporating a well-researched historic period into the analysis also reduces the uncertainty of the estimated discharge-frequency analysis. A comprehensive study of historical floods dates back to 1896 is included in the updated 2009 flood frequency curve.

Synthetic hydrology was developed to supplement the 83-years of data and add resolution to the 0.01 to the 0.001 annual exceedence probability (AEP) range. The synthetic analysis used unregulated gage data from throughout the basin to create a number of hypothetical storm patterns based on large historic storms, which were scaled to larger events than what has been seen in the basin during the recorded period. A HEC-ResSim model of the Cowlitz basin was used to route the hypothetical events through the reservoirs to determine likely regulated peak discharges at Castle Rock. The results were merged into a composite discharge-frequency relationship representing the mean frequency for a given discharge.

3.3. EXCEEDANCE PROBABILITY RESULTS FOR THE COWLITZ RIVER

The new regulated discharge-frequency curve at Castle Rock is graphically drawn, first through the 83 plotted points as describe earlier in the report, and then using the results from the synthetic analysis beyond that. The new regulated discharge-frequency curve at Castle Rock is shown in Figure 3-1. Updated regulated discharge-frequency curve for Cowlitz River at Castle Rock

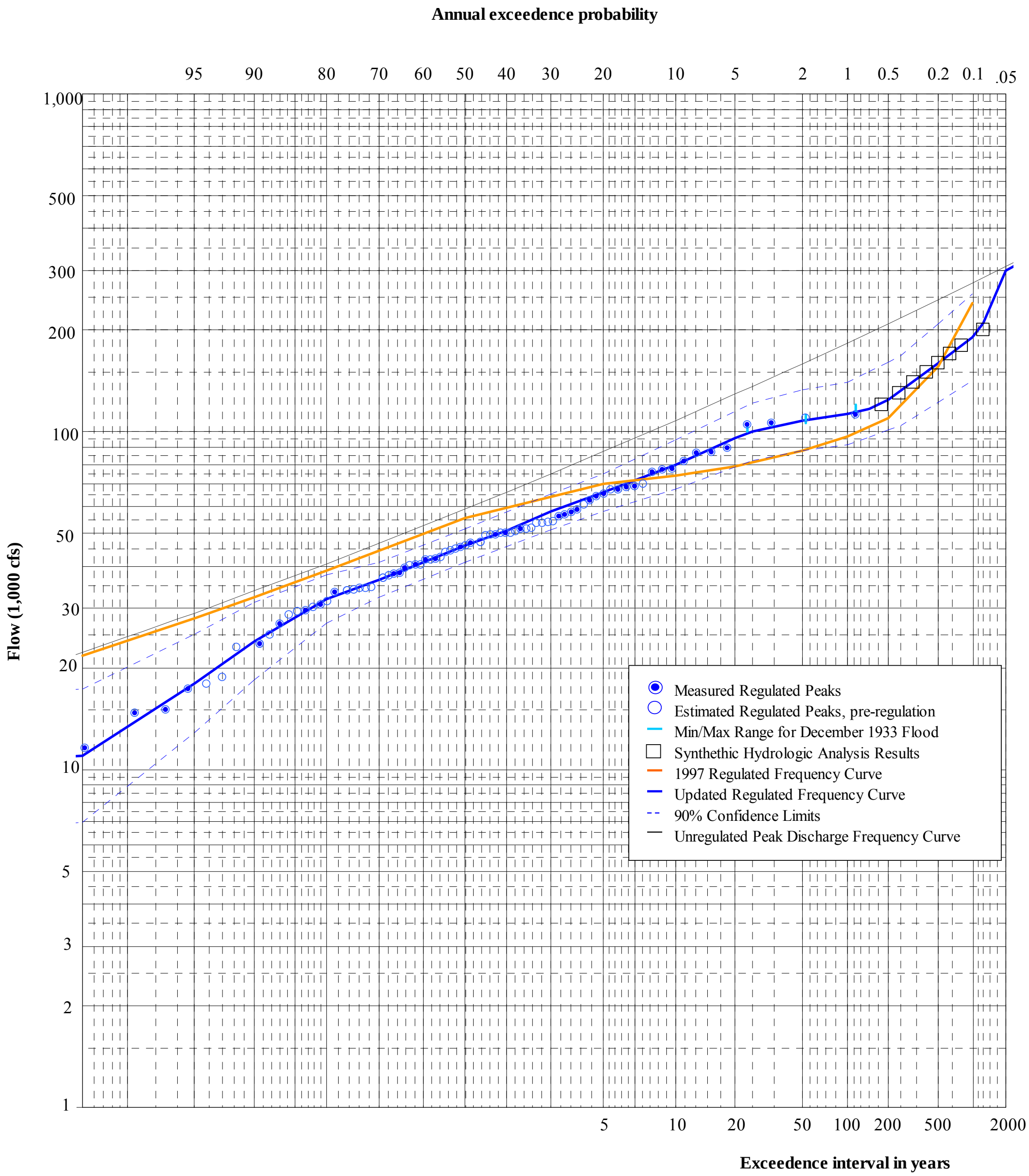

Figure 3-1. Updated regulated discharge-frequency curve for Cowlitz River at Castle Rock

3.4. HYDROLOGIC UNCERTAINTY

Uncertainty for the updated hydrology at Castle Rock is related to the length of the systematic record. An equivalent years-of-record (EYR) of 90 years is calculated based on 83 years of data adjusted for the historical period and for additional uncertainty from estimating regulated peaks for pre-regulation years. The additional equivalent years of record are due to knowledge of historic flood event prior to the beginning of annual record keeping. Investigation of historical floods on the lower Cowlitz indicates that the largest flood observed in the systematic record period (observed regulated event of 1996) is the largest event dating back to 1896. The additional 30 years of historical record increases certainty but at a

significantly reduced value, estimated at 50% for this analysis. An additional 15 years of EYR are added to the 75 years representing the observed period of record for a total of 90 years representing the EYR of this hydrologic study. The same EYR is assumed for the three flow-change locations below Castle Rock.

3.5. UPDATED PEAK DISCHARGES

Discharge-frequency curves for flow-change locations below Castle Rock are calculated from the regulated frequency curve at Castle Rock and discharge-frequency data from regional regression analyses of the major local tributaries. Tributary inputs are reduced by a constant percentage based on observed timing of peaks from historic events. The final discharge-frequency flows from Castle Rock to the Columbia River are shown in Table 3-1. More details regarding the hydrologic analysis is included in Appendix B.

Table 3-1. Regulated Peak Discharges for Cowlitz River at Castle Rock and at major tributary confluences below Castle Rock

Percent Chance Exceedance	Cowlitz River Peak Flow (cfs)			
	At Castle Rock RM 20.06	below Arkansas Creek RM 16.1	below Ostrander Creek RM 8.64	below Coweeman River RM 1.61
99.99	10,000	10,200	10,400	11,200
99	11,000	11,500	11,700	13,300
95	18,000	18,600	19,000	21,100
90	24,000	24,700	25,200	27,700
80	32,000	32,900	33,400	36,400
70	36,500	37,500	38,100	41,500
60	41,000	42,200	42,800	46,600
50	46,000	47,300	48,000	52,200
40	51,000	52,400	53,200	57,800
30	58,000	59,600	60,500	65,600
20	66,000	67,800	68,800	74,500
10	80,000	82,200	83,400	90,100
5	96,000	98,600	99,900	107,500
4	100,000	102,700	104,100	112,000
2	108,000	111,000	112,600	121,300
1	113,000	116,400	118,200	127,700
0.7	117,000	120,600	122,400	132,400
0.5	124,000	127,700	129,700	140,000
0.2	160,000	164,200	166,500	177,800
0.1	190,000	194,600	197,000	209,100
0.08	210,000	214,700	217,100	229,500
0.05	300,000	304,900	307,500	320,400
0.01	390,000	395,800	398,800	413,500

4. HYDRAULIC ANALYSIS

4.1. OVERVIEW

Hydraulic reliability is defined using stage-discharge curves with uncertainty. Stage-discharge curves are developed as part of the monitoring plan for the Mount St. Helens project and are an integral part the analysis used to determine levels of flood protection in areas protected by levees on the Cowlitz River. Changes in the Cowlitz River's channel capacity or the river's ability to pass a given discharge at a consistent stage are reflected in the stage discharge curves. Cowlitz River stages for a given annual exceedance event are determined through computer modeling of the Cowlitz River utilizing the most recent river bathymetry, floodplain topography and other structural features (levees, railroad embankments and bridges). Computed stages with uncertainty are developed for the annual exceedance floods presented in the Section 2.

4.2. STAGE-DISCHARGE CURVES AT COWLITZ RIVER INDEX LOCATIONS

Stage-discharge curves with uncertainty are developed at each index location within the four Cowlitz River levee projects included in this LOP analysis. The Corps of Engineers 1-D computer model HEC-RAS, version 4.0, is utilized to develop water surface profiles for the 23 exceedance events shown in Table 3-1. Calibration of this model is achieved using data obtained from a flooding event in January 2009. Observed stage data was collected at eleven (11) locations from river mile 1.61 to river mile 19.52. Exact locations of the observed calibration points and the alignment of the hydraulic cross sections are shown in Figure 1-2. Each index location shown in Table 2-1 is tied to a cross section from the hydraulic model and the computed results for all exceedance probabilities (0.1% to 99%) make up the stage-discharge curve at the particular index location. A detailed discussion of the development of this hydraulic model is included in Appendix C. Appendix C also contains the stage-discharge curve that corresponds to each index point listed in Table 2-1.

4.3. UNCERTAINTY IN COMPUTED STAGE-DISCHARGE CURVES

Computed stages from the HEC-RAS model represent mean or expected stages corresponding to a particular discharge and are used to define stage-discharge rating curves for each index location. Uncertainty in the stage-discharge curves is defined using a normal probability density function around computed stages at each index location, with the computed stage representing the median value in the normal distribution. As described in USACE technical guidance (EM 1110-2-1619, 1996) typical sources of uncertainty include hydraulic model and data limitations and natural variations as represented in gage data. The total uncertainty for these influences on the stage-discharge relation can be estimated as:

$$S_t = (S_{natural}^2 + S_{model}^2)^{0.5} \qquad \textit{Equation 1}$$

Where;

S_t is the standard deviation of the total uncertainty;

$S_{natural}$ is the natural uncertainty; and

S_{model} is the modeling uncertainty.

The supply of sediment from the Toutle watershed resulting from the 1980 Mount St. Helens eruption continues to affect the stage-discharge relationship in the Cowlitz River. To address the uncertainty

associated with single event sedimentation, Equation 4 is expanded to include the uncertainty due to sedimentation:

$$S_t = (S_{natural}^2 + S_{model}^2 + S_{sedimentation}^2)^{0.5} \qquad \textit{Equation 2}$$

Where;

S_t is the standard deviation of the total uncertainty;

$S_{natural}$ is the natural uncertainty;

S_{model} is the modeling uncertainty; and

$S_{sedimentation}$ is the uncertainty due to gain or loss of channel capacity due to sedimentation.

In general, the standard deviation of stage uncertainty would be expected to increase with a decrease in data availability, accuracy, and model calibration/validation results. In Equation 5, each component of the total standard deviation represents the summation of the individual random statistics describing each of the major components which are assumed to be normally distributed.

4.3.1. Natural Uncertainty

Uncertainty in the computed stage-discharge curves due to natural variation is developed using sensitivity analysis in the HEC-RAS computer model of the Cowlitz River. Natural variability is represented in the HEC-RAS computer model as variations in Manning's roughness coefficient. Uncertainty in the computed water surface elevations is computed by using a 30 percent variation in Manning's roughness coefficient from RM 0.01 to RM 8.11 and a 14 percent variation in Manning's roughness from RM 8.11 to RM 20.06. This variation in Manning's roughness is considered to represent four standard deviations and therefore was divided by 4 to obtain the final uncertainty in stage. Results from the Natural uncertainty computations are included in Appendix C.

4.3.2. Model Uncertainty

Model uncertainty in the stage-discharge curve incorporates that uncertainty involved with the model selection and the overall ability of the particular computer model to describe actual water surface profiles. The roughness values that were used to calibrate the computed water surface profile in the steady state model were verified with gage data, measured high water marks, and computed theoretical roughness values. For this reason model variation from Table 5-2 of EM 1110-2-1619 is based on a *good* estimate of Manning's n reliability. As a result, the model uncertainty is set uniformly to 0.3 ft.

4.3.3. Sediment Uncertainty

The supply of sediment from the Toutle watershed resulting from the 1980 Mount St. Helens eruption continues to affect the stage-discharge relationship in the Cowlitz River. Movement of channel bedforms and high sediment loads during storm events introduce additional uncertainty in the computed water surface profiles. To address the added uncertainty associated with single event sedimentation an additional uncertainty term is added to the total uncertainty. Computation of the sediment uncertainty was accomplished by analyzing gage data, reviewing theoretical relationships for bedform formation, and reviewing historical survey information. A complete explanation of this analysis is included in Appendix C. Ultimately, the sediment uncertainty was estimated to be 0.7 ft downstream of Castle Rock and 0.25 ft upstream of Castle Rock.

4.3.4. Total Uncertainty

Sediment uncertainty combined with natural and model uncertainty, using Equation 5, resulted in the total uncertainty values that are used in FDA's Monte Carlo routine. Total variances as well as the stage discharge rating curves at each index point are included in Appendix C. In general, the total uncertainty ranges from 0.5 ft to 1.3 ft and increases with decreasing frequency profiles. The stage discharge rating curves, combined with the levee and the hydrology data, provide a complete input dataset used to create an FDA model of the Cowlitz levee level of protection.

5. CONCLUSION

5.1. OVERVIEW

FDA runs are compiled using the Levee data, Hydrology data, and Hydraulic data presented in Sections 2, 3, and 4, respectively. No economic analysis is included in the FDA modeling of the Cowlitz Levees. Results from FDA are reported in terms of a non-exceedance probability (or assurance) of containing a given %-chance exceedance event. The level of protection at a given index point is determined based on the exceedance event that would result in a 90 percent assurance that the levee would withstand the event. Exceedance probabilities are linearly interpolated where the 90 percent chance was not directly output. Finally, the overall level of protection for the particular levee system corresponds to the index point for the respective system that has the lowest level of protection. All input data for the FDA computer model are included in the form of screen captures in Appendix A.

5.2. CURRENT LEVEL OF FLOOD PROTECTION

Before the level of protection analysis was completed in FDA the compiled hydraulic model results were compared to the levee assessments at the discrete index point locations. Table 5-1 shows the results from the steady state HEC-RAS model of the Cowlitz River compared to the Levee Assessment information presented in Section 2. Table 5-1 is provided to preliminarily show where the computed water surface elevations encroach into failure zone of the levees. For example, Castle Rock index point CRIP 2 and Lexington index point LXIP 1 both show that the water surface elevation is encroached into or very close to the zone of levee failure. While the information presented in Table 5-1 is useful in comparing the computed water surface elevations to the top of levee and safe water levels, it does not incorporate the full uncertainty and the probabilistic nature of the FDA analysis and is not any indication of level of protection.

Table 5-1. Hydraulic Model Results Compared to Levee Assessments

Index Point	River Mile	Authorized LOP	Computed WSEL at Authorized LOP*	Total Hydraulic Variance	Safe Water Level	Top of Levee
			(ft, NAVD 88)	(ft)	(ft, NAVD 88)	(ft, NAVD 88)
Castle Rock Levee						
CRIP 1	17.42	1/118	58.22	0.99	65.8	65.8
CRIP 2	17.00	1/118	56.15	0.97	57.3	60.9
CRIP 3	15.91	1/118	54.20	0.86	58.5	58.5
Lexington Levee						
LXIP 1	8.64	1/167	37.77	1.14	38.2	45.7
LXIP 2	8.30	1/167	35.70	1.20	42.6	42.6
Longview Levee						
LVIP 1	4.90	1/167	26.75	1.13	35.1	35.1
LVIP 2	4.68	1/167	26.95	1.07	34.8	37.4
LVIP 3	3.59	1/167	25.68	0.96	32.8	32.8
LVIP 4	3.27	1/167	24.93	0.94	32.0	32.5

Kelso Levee						
KLIP 1	7.00	1/143	30.60	1.27	37.7	37.7
KLIP 2	6.19	1/143	29.36	1.15	37.4	40.3
KLIP 3	4.02	1/143	26.29	0.98	33.5	34.5
KLIP 4	3.70	1/143	25.87	0.96	30.4	33.4

* Computed WSELs are obtained directly from the Cowlitz River HEC-RAS steady state model.

The current LOP (May 2008) for the Cowlitz River levees is summarized in Table 5-2. These results are based on the conditional non-exeedance probabilities in the Project Performance output table from the HEC-FDA analysis. The current LOP numbers in Table 5-2 are based on the flood event in which the levee has a 90% probability to contain without failing. Table 5-2 also summarizes the probability of the levees to contain the 1/250-year and 1/100-year events.

Table 5-2. Cowlitz River levees, level of flood protection in May 2008.

Index Point		RM	Top of Levee (ft)	Safe Water Level	Authorized LOP	Discharge at LOP (cfs)	Expected Stage at Authorized LOP (ft)	Current LOP (1/years)	Probability of Containing the 0.4% (1/250-yr) Annual Change Exceedance Flood	Probability of Containing the 1.0% (1/100-yr) Annual Change Exceedance Flood
Castle Rock 1	CRIP 1	17.42	65.8	65.8	118	115,034	58.22	468	99.7%	100.0%
Castle Rock 2	CRIP 2	17	57.3	60.9	118	115,034	56.15	109	68.3%	93.5%
Castle Rock 3	CRIP 3	15.91	58.5	58.5	118	118,536	54.20	160	84.6%	99.1%
Lexington 1	LXIP 1	8.64	38.2	45.7	167	126,094	37.77	202	88.5%	97.8%
Lexington 2	LXIP 2	8.3	42.6	42.6	167	126,094	35.70	326	98.8%	100.0%
Kelso 1	KLIP 1	7	37.7	37.7	143	122,426	30.60	>500	99.7%	100.0%
Kelso 2	KLIP 2	6.19	37.4	40.3	143	122,426	29.36	>500	100.0%	100.0%
Keslo 3	KLIP 3	4.02	33.5	34.5	143	122,426	26.29	>500	100.0%	100.0%
Keslo 4	KLIP 4	3.7	30.4	33.4	143	122,426	25.87	470	99.4%	100.0%
Longview 1	LVIP 1	4.9	35.1	35.1	167	126,094	26.75	>500	100.0%	100.0%
Longview 2	LVIP 2	4.68	34.8	37.4	167	126,094	26.95	>500	100.0%	100.0%
Longview 3	LVIP 3	3.59	32.8	32.8	167	126,094	25.68	>500	100.0%	100.0%
Longview 4	LVIP 4	3.27	32.0	32.5	167	126,094	24.93	>500	100.0%	100.0%

Note: All Elevations given in ft NAVD88

Final reported LOP is based on the index point in the levee system with lowest LOP. For Castle Rock the lowest LOP occurs at CRIP 2 where the current computed LOP is 1/80 years. Table 5-3 summarizes the final level of protection determined from the data presented in
Table 5-2.

Table 5-3. 2009 Level of Protection for each Levee System

Levee	Authorized LOP	Current LOP
Castle Rock	118	109
Lexington	167	202
Kelso	143	470
Longview	167	>500

6. LITERATURE CITED

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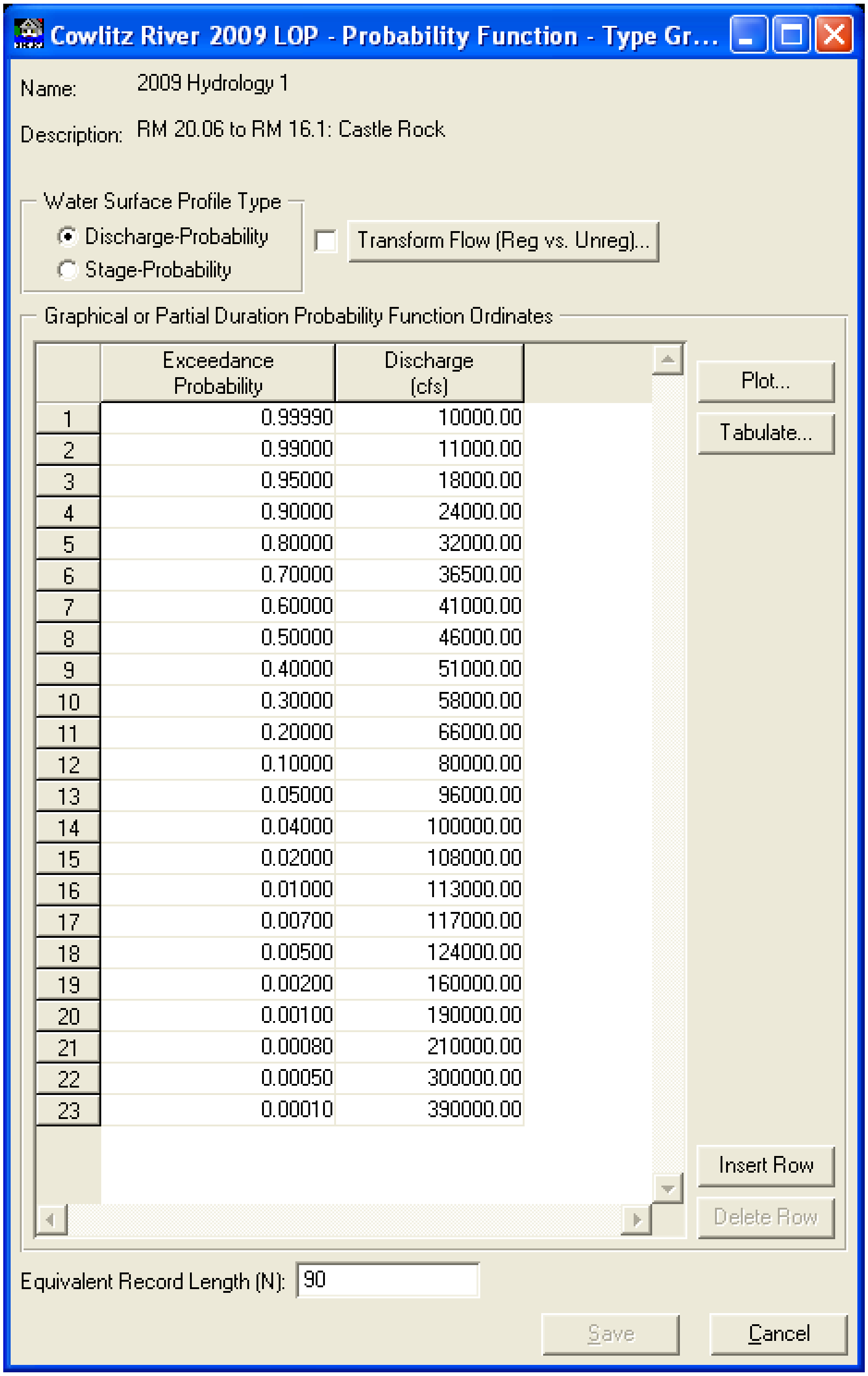
Appendix A. Computer Model HEC FDA Inputs

A.1. Flow Frequency Curves


Cowlitz River 2009 LOP - Probability Function - Type Gr...

Name: 2009 Hydrology 1

Description: RM 20.06 to RM 16.1: Castle Rock

Water Surface Profile Type
Discharge-Probability
Stage-Probability
Transform Flow (Reg vs. Unreg)...

Graphical or Partial Duration Probability Function Ordinates

	Exceedance Probability	Discharge (cfs)
1	0.99990	10000.00
2	0.99000	11000.00
3	0.95000	18000.00
4	0.90000	24000.00
5	0.80000	32000.00
6	0.70000	36500.00
7	0.60000	41000.00
8	0.50000	46000.00
9	0.40000	51000.00
10	0.30000	58000.00
11	0.20000	66000.00
12	0.10000	80000.00
13	0.05000	96000.00
14	0.04000	100000.00
15	0.02000	108000.00
16	0.01000	113000.00
17	0.00700	117000.00
18	0.00500	124000.00
19	0.00200	160000.00
20	0.00100	190000.00
21	0.00080	210000.00
22	0.00050	300000.00
23	0.00010	390000.00

Plot...
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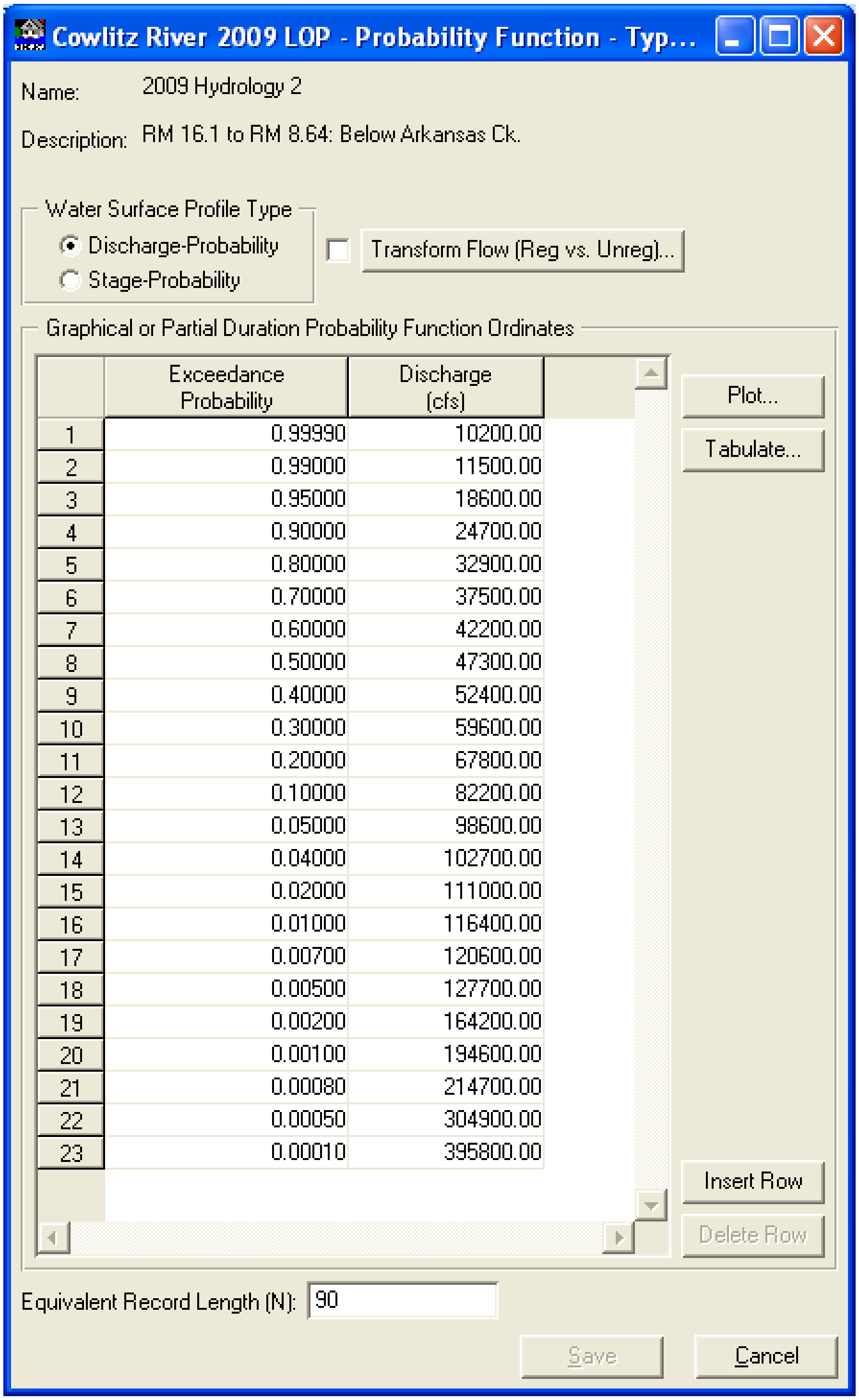
Equivalent Record Length (N): 90

Save
Cancel

Figure A. 1. FDA Hydrology Curve 1



Cowlitz River 2009 LOP - Probability Function - Typ...

Name: 2009 Hydrology 2

Description: RM 16.1 to RM 8.64: Below Arkansas Ck.

Water Surface Profile Type

(•) Discharge-Probability

( ) Stage-Probability

[ ] Transform Flow (Reg vs. Unreg)...

Graphical or Partial Duration Probability Function Ordinates

	Exceedance Probability	Discharge (cfs)
1	0.99990	10200.00
2	0.99000	11500.00
3	0.95000	18600.00
4	0.90000	24700.00
5	0.80000	32900.00
6	0.70000	37500.00
7	0.60000	42200.00
8	0.50000	47300.00
9	0.40000	52400.00
10	0.30000	59600.00
11	0.20000	67800.00
12	0.10000	82200.00
13	0.05000	98600.00
14	0.04000	102700.00
15	0.02000	111000.00
16	0.01000	116400.00
17	0.00700	120600.00
18	0.00500	127700.00
19	0.00200	164200.00
20	0.00100	194600.00
21	0.00080	214700.00
22	0.00050	304900.00
23	0.00010	395800.00

Plot...

Tabulate...

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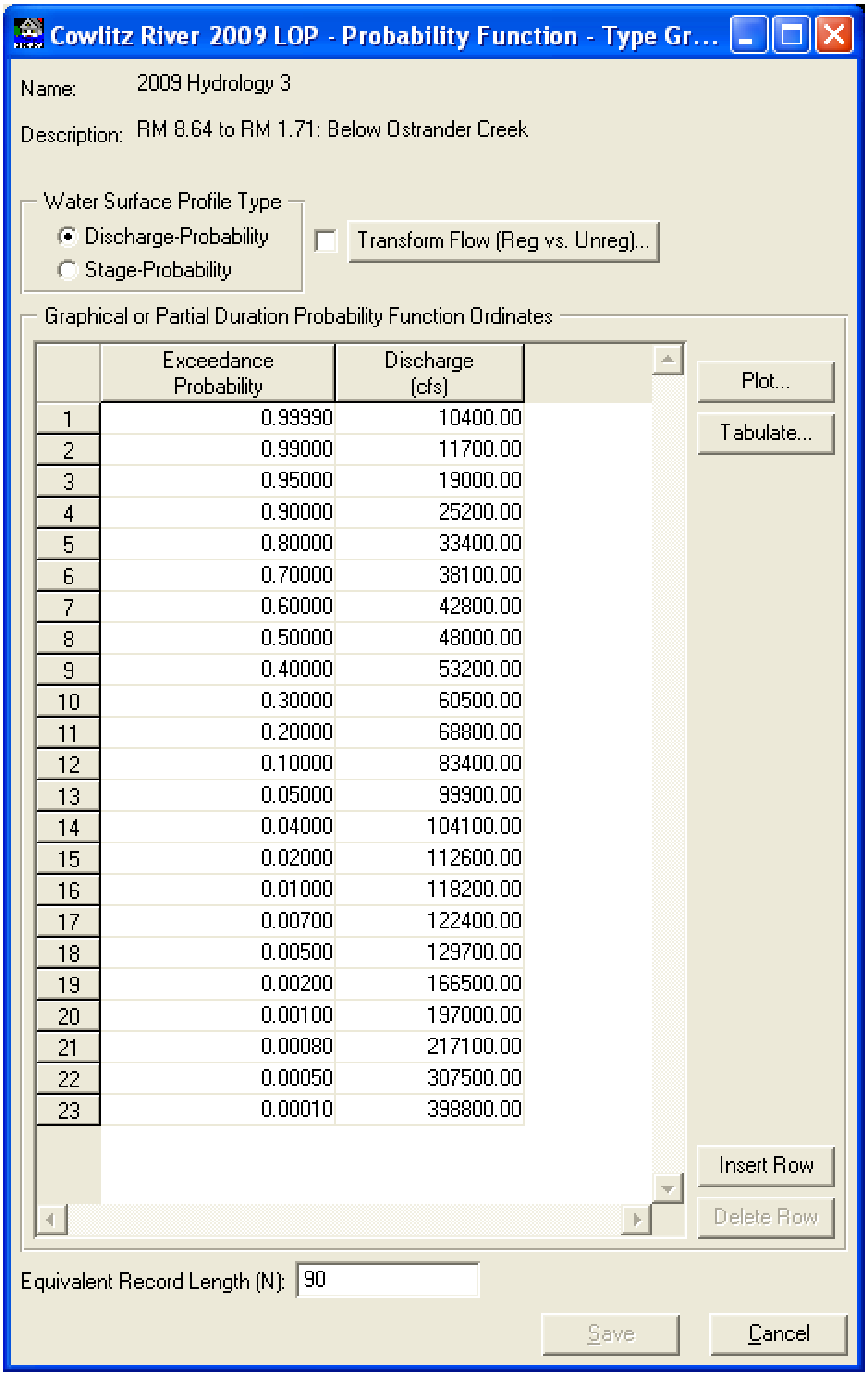
Equivalent Record Length (N): 90

Save

Cancel

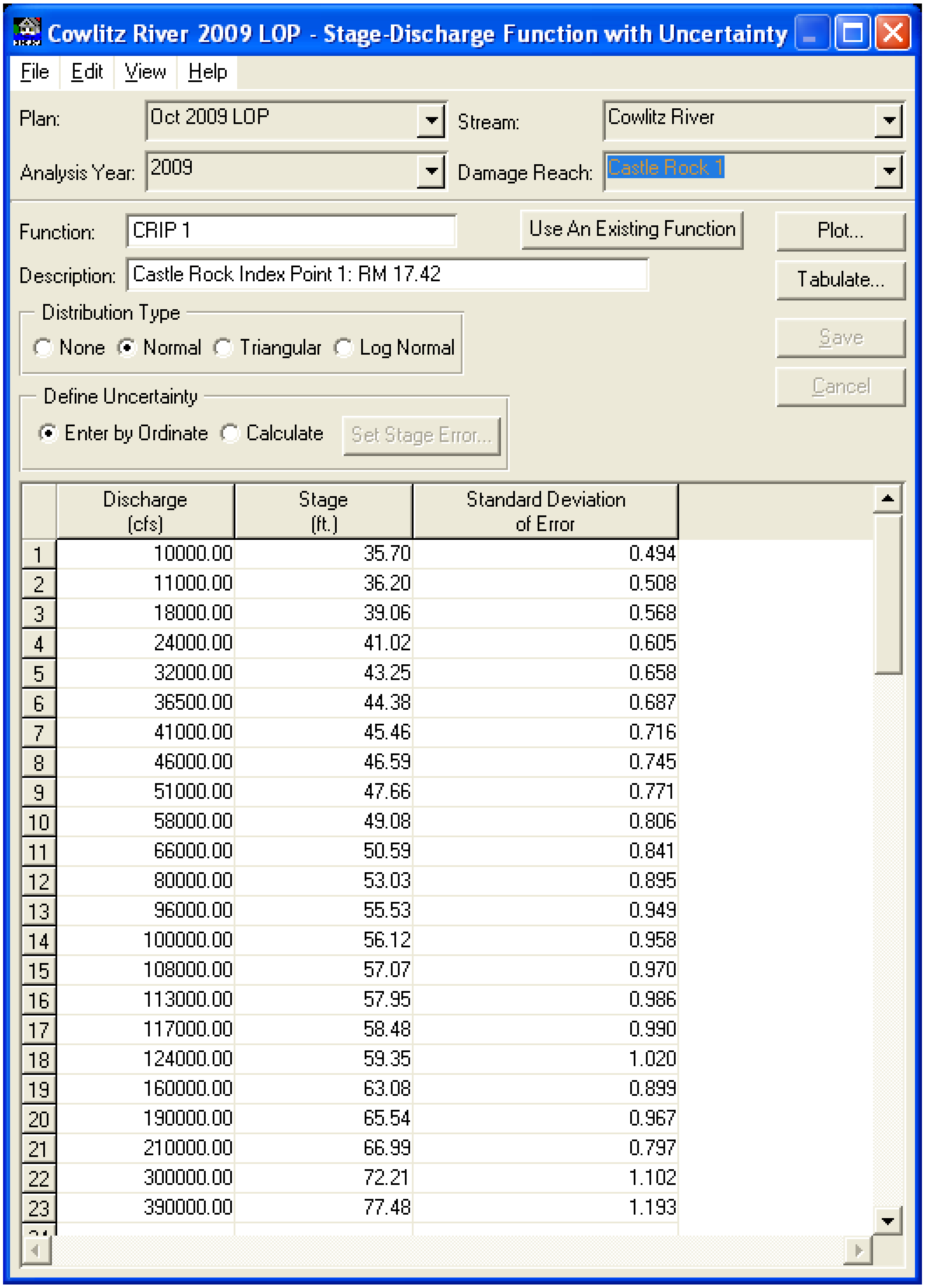
Figure A. 2. FDA Hydrology Curve 2



	Exceedance Probability	Discharge (cfs)
1	0.99990	10400.00
2	0.99000	11700.00
3	0.95000	19000.00
4	0.90000	25200.00
5	0.80000	33400.00
6	0.70000	38100.00
7	0.60000	42800.00
8	0.50000	48000.00
9	0.40000	53200.00
10	0.30000	60500.00
11	0.20000	68800.00
12	0.10000	83400.00
13	0.05000	99900.00
14	0.04000	104100.00
15	0.02000	112600.00
16	0.01000	118200.00
17	0.00700	122400.00
18	0.00500	129700.00
19	0.00200	166500.00
20	0.00100	197000.00
21	0.00080	217100.00
22	0.00050	307500.00
23	0.00010	398800.00

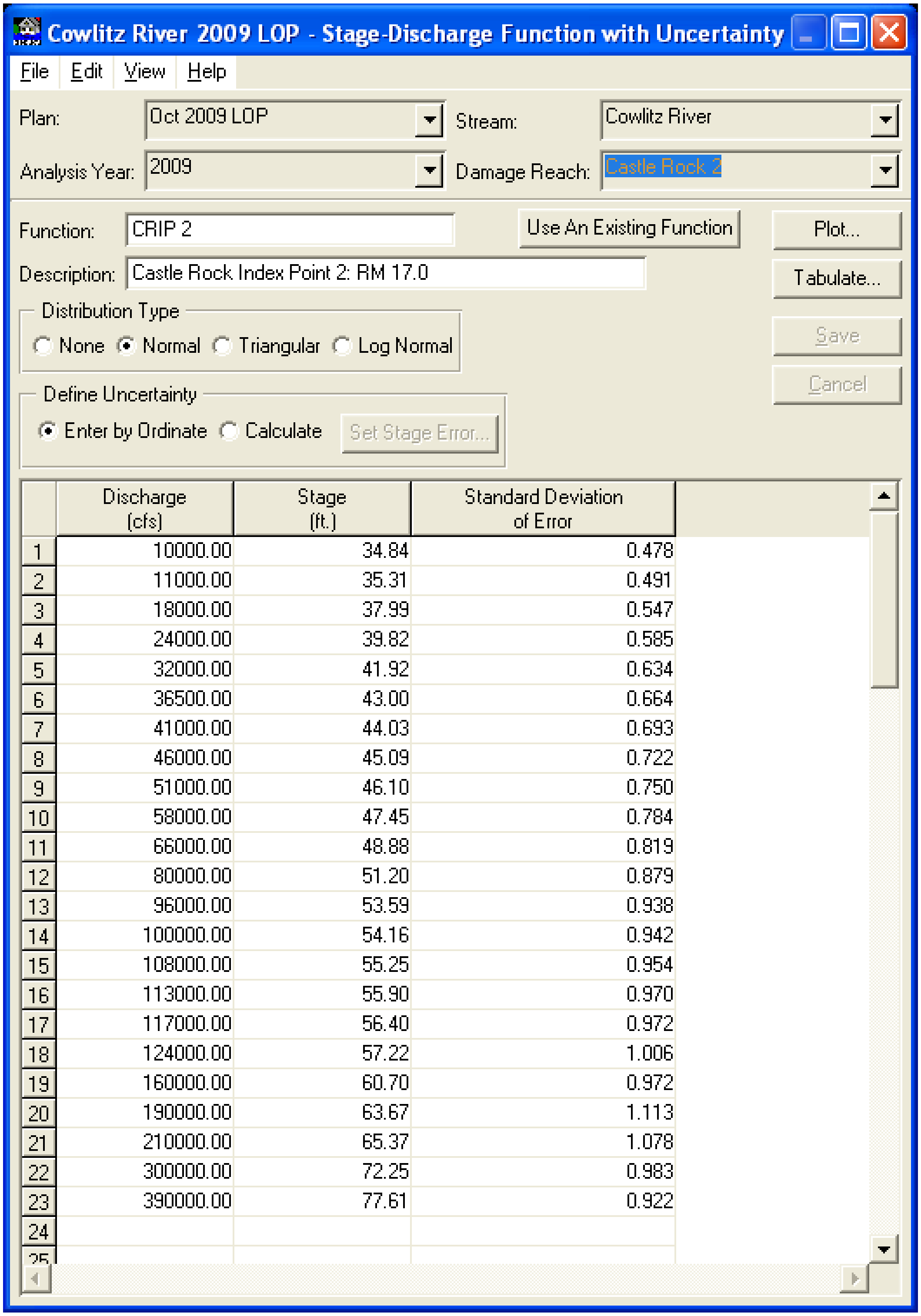
Figure A. 3. FDA Hydrology Curve 3

A.2. Stage Discharge Curves



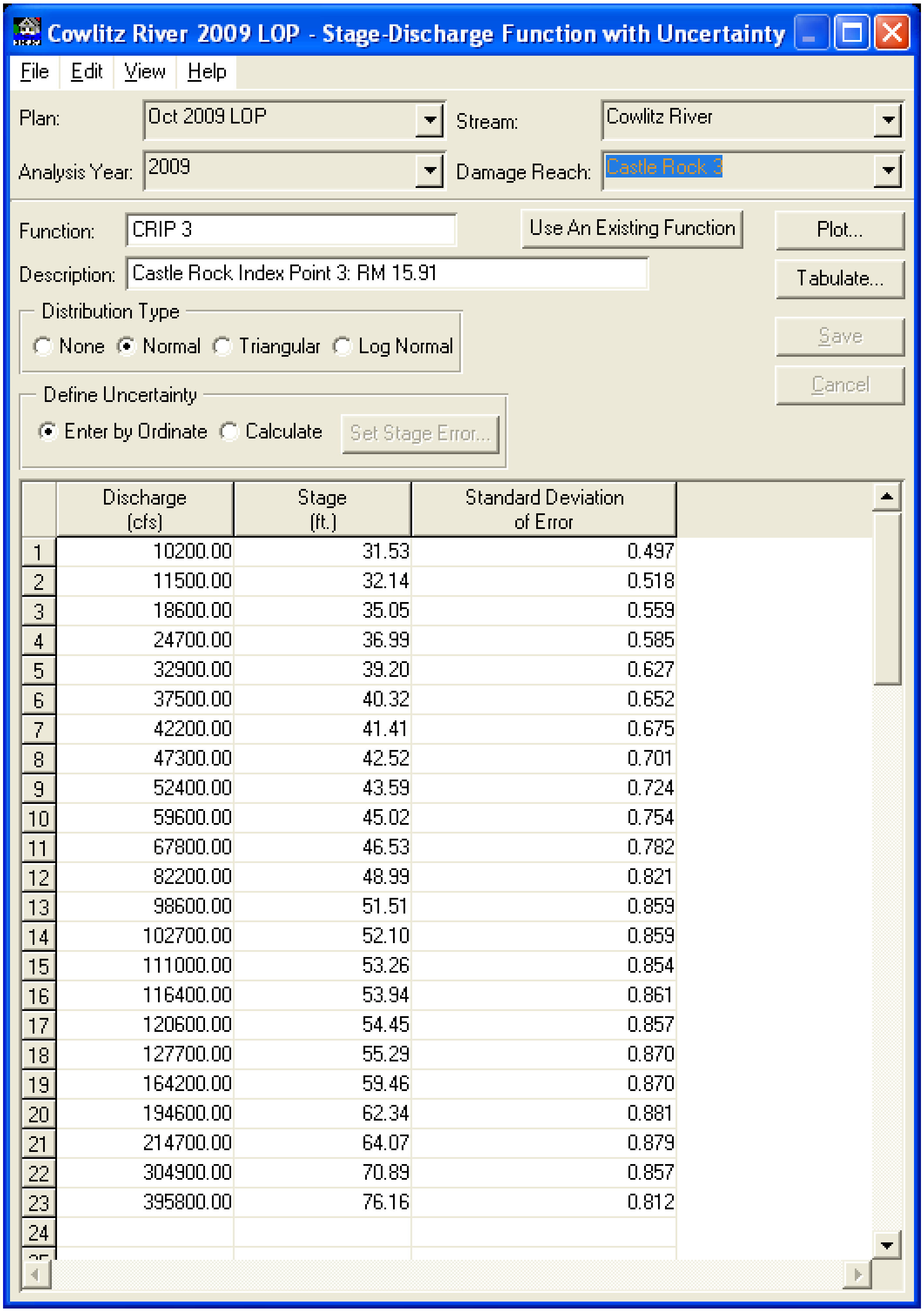
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	10000.00	35.70	0.494
2	11000.00	36.20	0.508
3	18000.00	39.06	0.568
4	24000.00	41.02	0.605
5	32000.00	43.25	0.658
6	36500.00	44.38	0.687
7	41000.00	45.46	0.716
8	46000.00	46.59	0.745
9	51000.00	47.66	0.771
10	58000.00	49.08	0.806
11	66000.00	50.59	0.841
12	80000.00	53.03	0.895
13	96000.00	55.53	0.949
14	100000.00	56.12	0.958
15	108000.00	57.07	0.970
16	113000.00	57.95	0.986
17	117000.00	58.48	0.990
18	124000.00	59.35	1.020
19	160000.00	63.08	0.899
20	190000.00	65.54	0.967
21	210000.00	66.99	0.797
22	300000.00	72.21	1.102
23	390000.00	77.48	1.193

Figure A. 4. Stage Discharge Curve for Castle Rock 1 (CRIP1)



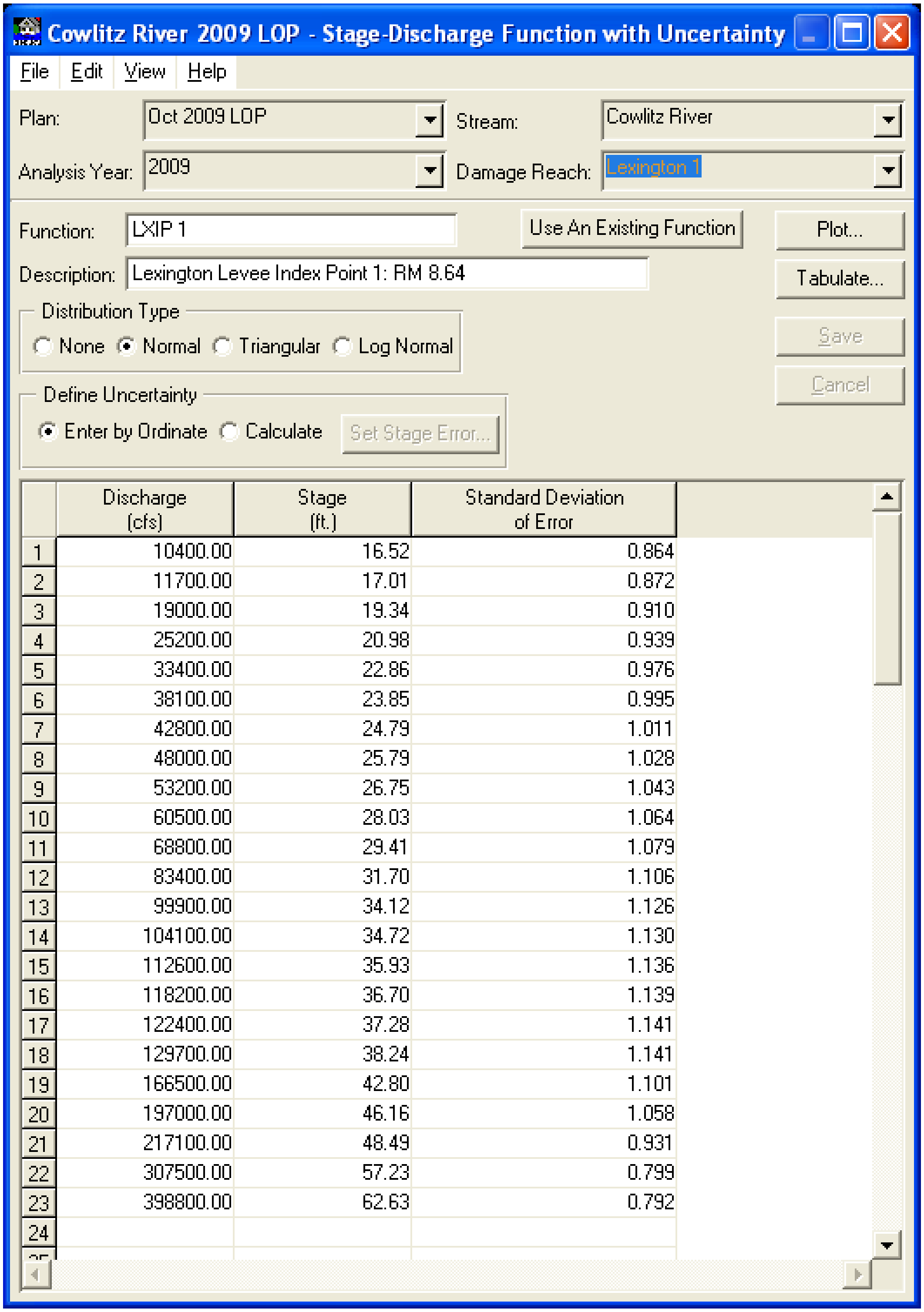
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	10000.00	34.84	0.478
2	11000.00	35.31	0.491
3	18000.00	37.99	0.547
4	24000.00	39.82	0.585
5	32000.00	41.92	0.634
6	36500.00	43.00	0.664
7	41000.00	44.03	0.693
8	46000.00	45.09	0.722
9	51000.00	46.10	0.750
10	58000.00	47.45	0.784
11	66000.00	48.88	0.819
12	80000.00	51.20	0.879
13	96000.00	53.59	0.938
14	100000.00	54.16	0.942
15	108000.00	55.25	0.954
16	113000.00	55.90	0.970
17	117000.00	56.40	0.972
18	124000.00	57.22	1.006
19	160000.00	60.70	0.972
20	190000.00	63.67	1.113
21	210000.00	65.37	1.078
22	300000.00	72.25	0.983
23	390000.00	77.61	0.922
24			
25			

Figure A. 5. Stage Discharge Curve for Castle Rock 2 (CRIP2)



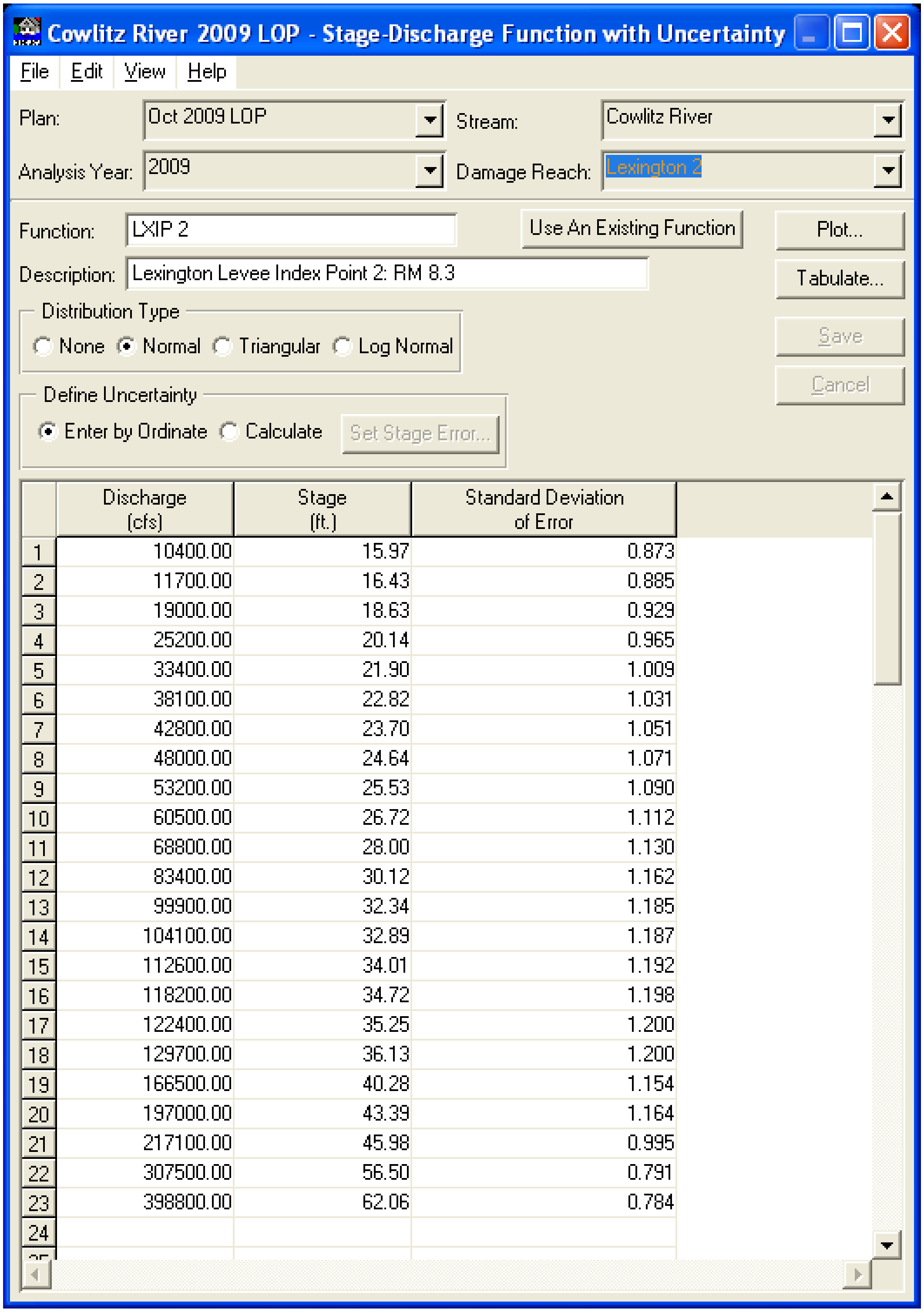
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	10200.00	31.53	0.497
2	11500.00	32.14	0.518
3	18600.00	35.05	0.559
4	24700.00	36.99	0.585
5	32900.00	39.20	0.627
6	37500.00	40.32	0.652
7	42200.00	41.41	0.675
8	47300.00	42.52	0.701
9	52400.00	43.59	0.724
10	59600.00	45.02	0.754
11	67800.00	46.53	0.782
12	82200.00	48.99	0.821
13	98600.00	51.51	0.859
14	102700.00	52.10	0.859
15	111000.00	53.26	0.854
16	116400.00	53.94	0.861
17	120600.00	54.45	0.857
18	127700.00	55.29	0.870
19	164200.00	59.46	0.870
20	194600.00	62.34	0.881
21	214700.00	64.07	0.879
22	304900.00	70.89	0.857
23	395800.00	76.16	0.812
24			

Figure A. 6. Stage Discharge Curve for Castle Rock 3 (CRIP3)



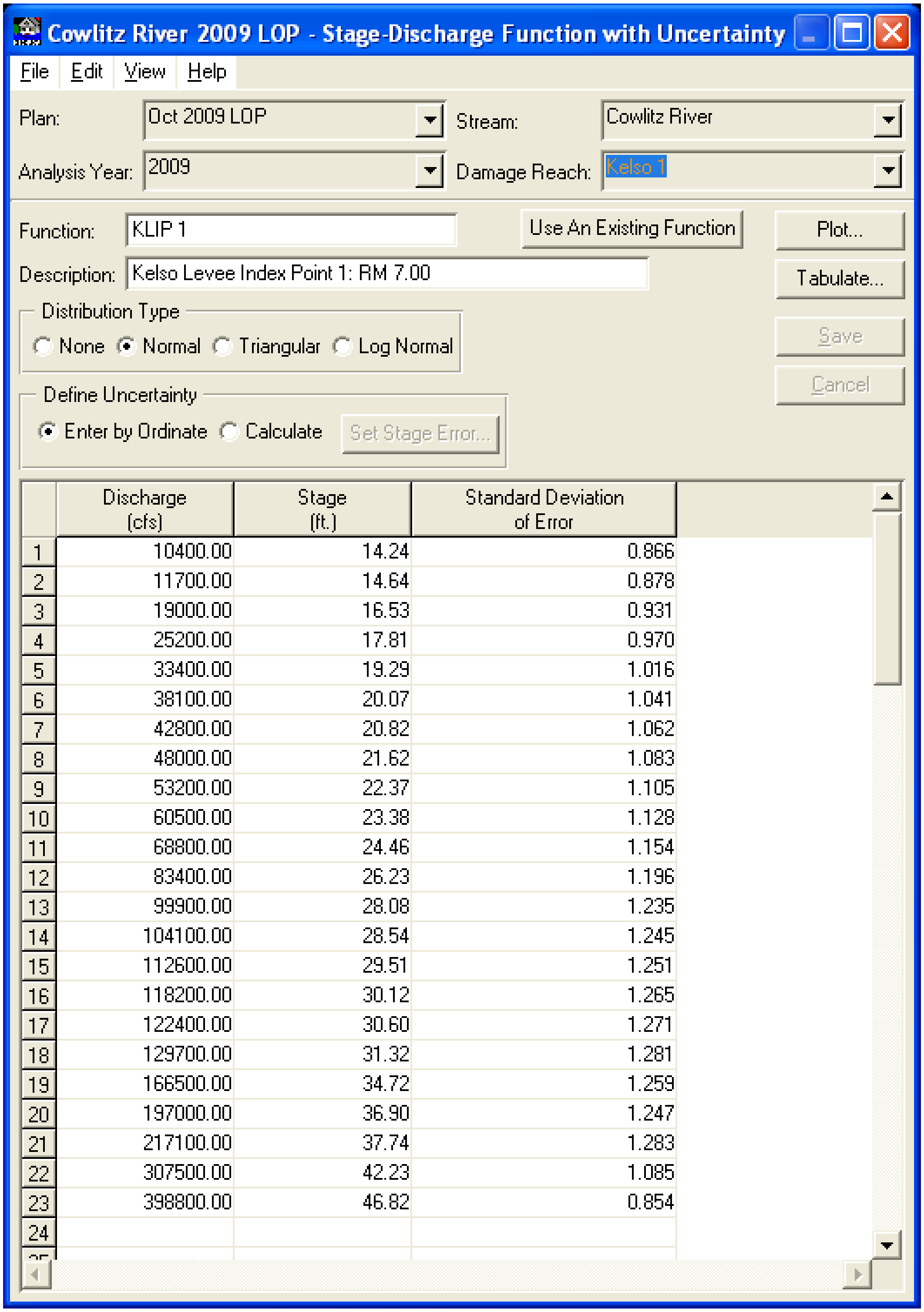
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	10400.00	16.52	0.864
2	11700.00	17.01	0.872
3	19000.00	19.34	0.910
4	25200.00	20.98	0.939
5	33400.00	22.86	0.976
6	38100.00	23.85	0.995
7	42800.00	24.79	1.011
8	48000.00	25.79	1.028
9	53200.00	26.75	1.043
10	60500.00	28.03	1.064
11	68800.00	29.41	1.079
12	83400.00	31.70	1.106
13	99900.00	34.12	1.126
14	104100.00	34.72	1.130
15	112600.00	35.93	1.136
16	118200.00	36.70	1.139
17	122400.00	37.28	1.141
18	129700.00	38.24	1.141
19	166500.00	42.80	1.101
20	197000.00	46.16	1.058
21	217100.00	48.49	0.931
22	307500.00	57.23	0.799
23	398800.00	62.63	0.792
24			

Figure A. 7. Stage Discharge Curve for Lexington 1 (LXIP1)



	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	10400.00	15.97	0.873
2	11700.00	16.43	0.885
3	19000.00	18.63	0.929
4	25200.00	20.14	0.965
5	33400.00	21.90	1.009
6	38100.00	22.82	1.031
7	42800.00	23.70	1.051
8	48000.00	24.64	1.071
9	53200.00	25.53	1.090
10	60500.00	26.72	1.112
11	68800.00	28.00	1.130
12	83400.00	30.12	1.162
13	99900.00	32.34	1.185
14	104100.00	32.89	1.187
15	112600.00	34.01	1.192
16	118200.00	34.72	1.198
17	122400.00	35.25	1.200
18	129700.00	36.13	1.200
19	166500.00	40.28	1.154
20	197000.00	43.39	1.164
21	217100.00	45.98	0.995
22	307500.00	56.50	0.791
23	398800.00	62.06	0.784
24			

Figure A. 8. Stage Discharge Curve for Lexington 2 (LXIP2)



Cowlitz River 2009 LOP - Stage-Discharge Function with Uncertainty

File Edit View Help

Plan: Oct 2009 LOP
Stream: Cowlitz River
Analysis Year: 2009
Damage Reach: Kelso 1

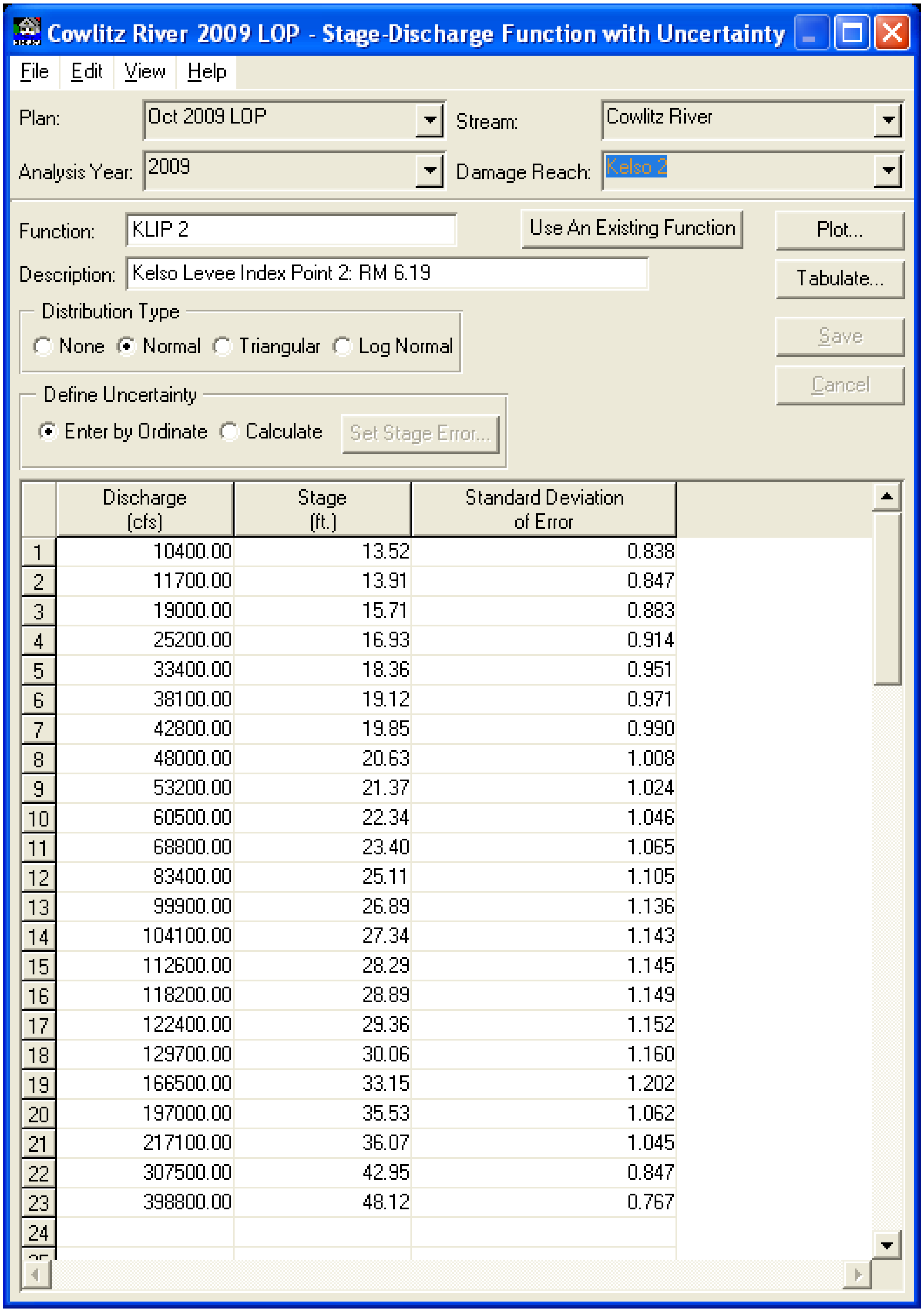
Function: KLIP 1
Use An Existing Function
Plot...
Description: Kelso Levee Index Point 1: RM 7.00
Tabulate...
Save
Cancel

Distribution Type
None Normal Triangular Log Normal

Define Uncertainty
Enter by Ordinate Calculate Set Stage Error...

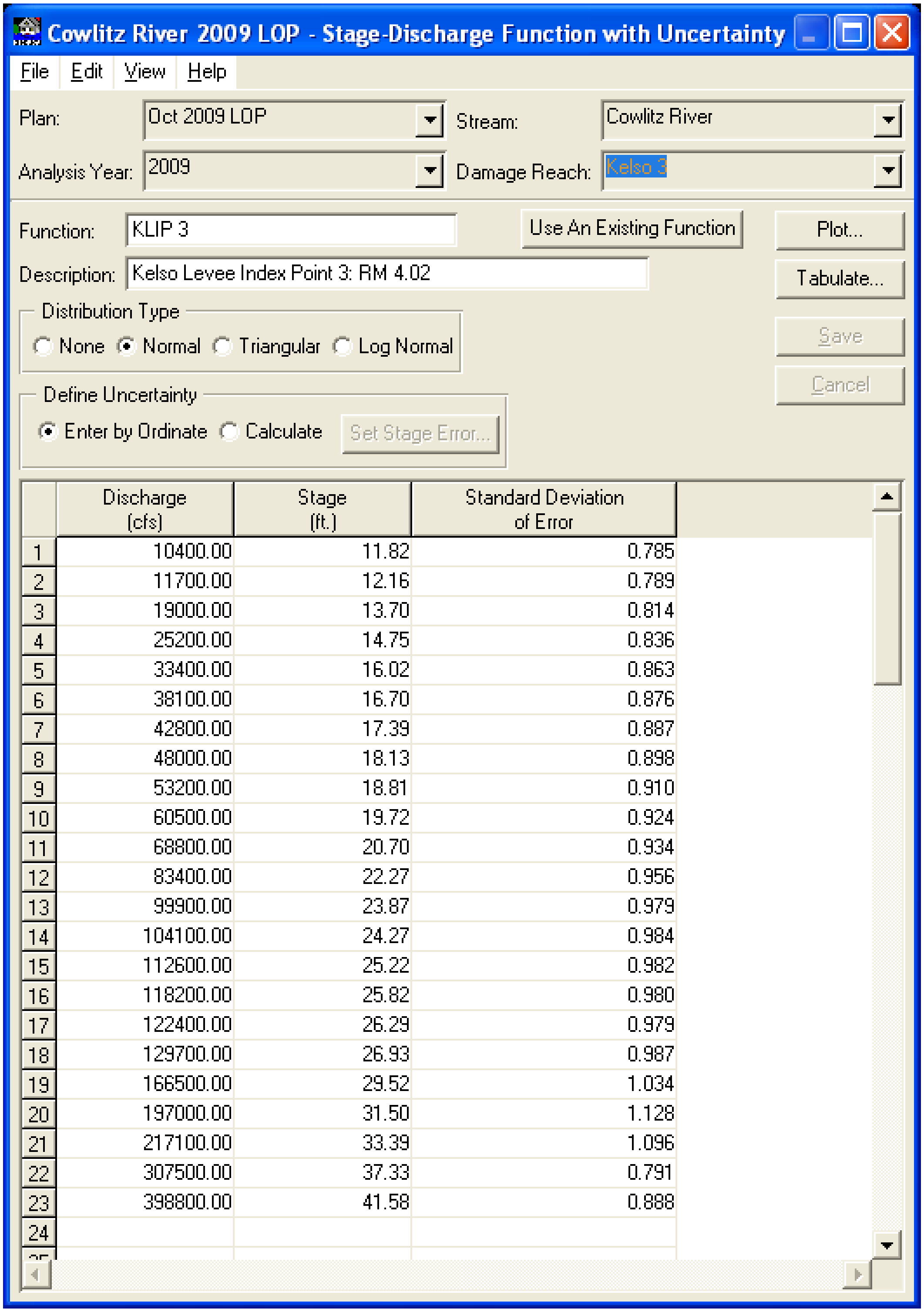
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	10400.00	14.24	0.866
2	11700.00	14.64	0.878
3	19000.00	16.53	0.931
4	25200.00	17.81	0.970
5	33400.00	19.29	1.016
6	38100.00	20.07	1.041
7	42800.00	20.82	1.062
8	48000.00	21.62	1.083
9	53200.00	22.37	1.105
10	60500.00	23.38	1.128
11	68800.00	24.46	1.154
12	83400.00	26.23	1.196
13	99900.00	28.08	1.235
14	104100.00	28.54	1.245
15	112600.00	29.51	1.251
16	118200.00	30.12	1.265
17	122400.00	30.60	1.271
18	129700.00	31.32	1.281
19	166500.00	34.72	1.259
20	197000.00	36.90	1.247
21	217100.00	37.74	1.283
22	307500.00	42.23	1.085
23	398800.00	46.82	0.854
24			

Figure A. 9. Stage Discharge Curve for Kelso 1 (KLIP1)



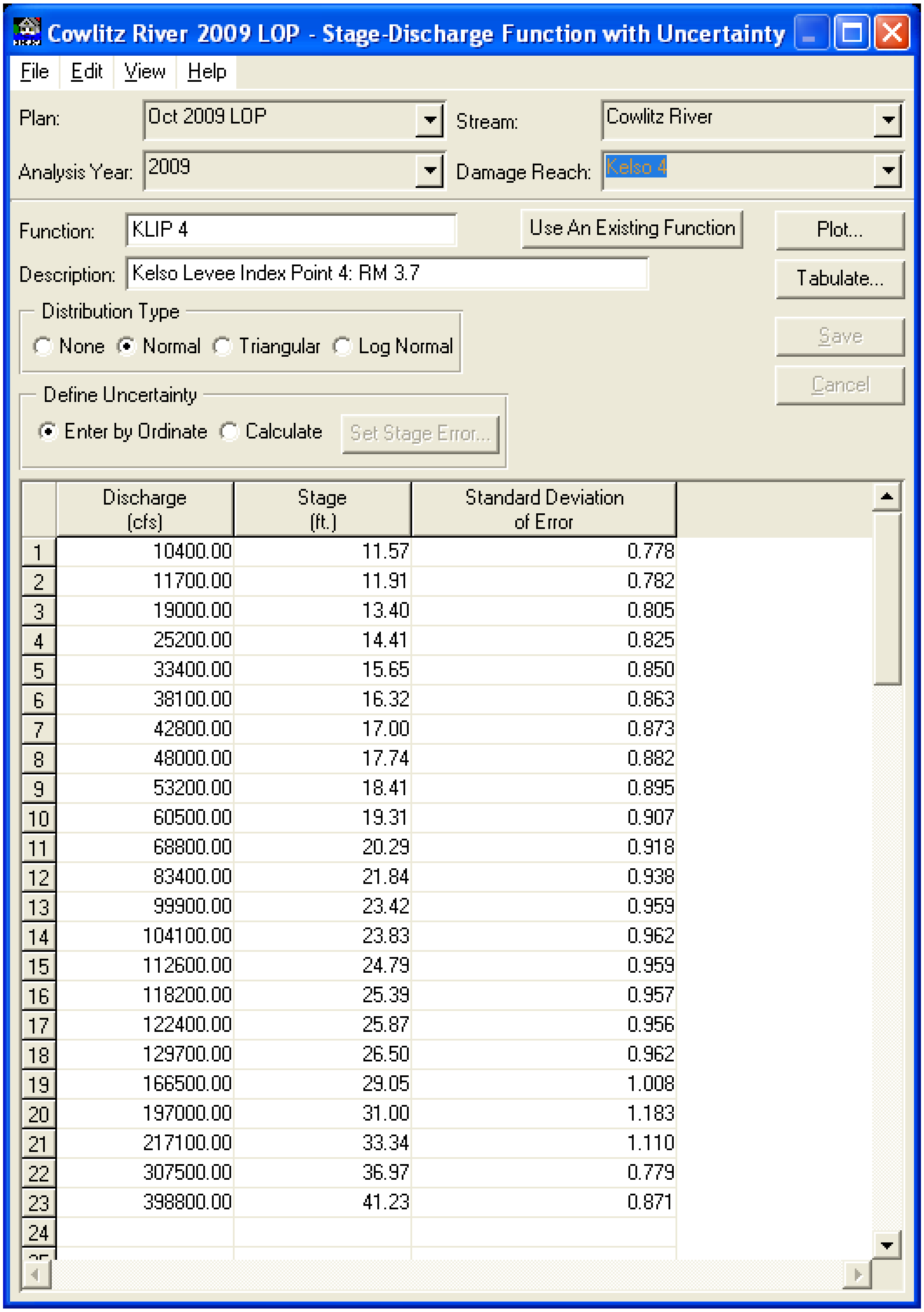
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	10400.00	13.52	0.838
2	11700.00	13.91	0.847
3	19000.00	15.71	0.883
4	25200.00	16.93	0.914
5	33400.00	18.36	0.951
6	38100.00	19.12	0.971
7	42800.00	19.85	0.990
8	48000.00	20.63	1.008
9	53200.00	21.37	1.024
10	60500.00	22.34	1.046
11	68800.00	23.40	1.065
12	83400.00	25.11	1.105
13	99900.00	26.89	1.136
14	104100.00	27.34	1.143
15	112600.00	28.29	1.145
16	118200.00	28.89	1.149
17	122400.00	29.36	1.152
18	129700.00	30.06	1.160
19	166500.00	33.15	1.202
20	197000.00	35.53	1.062
21	217100.00	36.07	1.045
22	307500.00	42.95	0.847
23	398800.00	48.12	0.767
24			

Figure A. 10. Stage Discharge Curve for Kelso 2 (KLIP2)



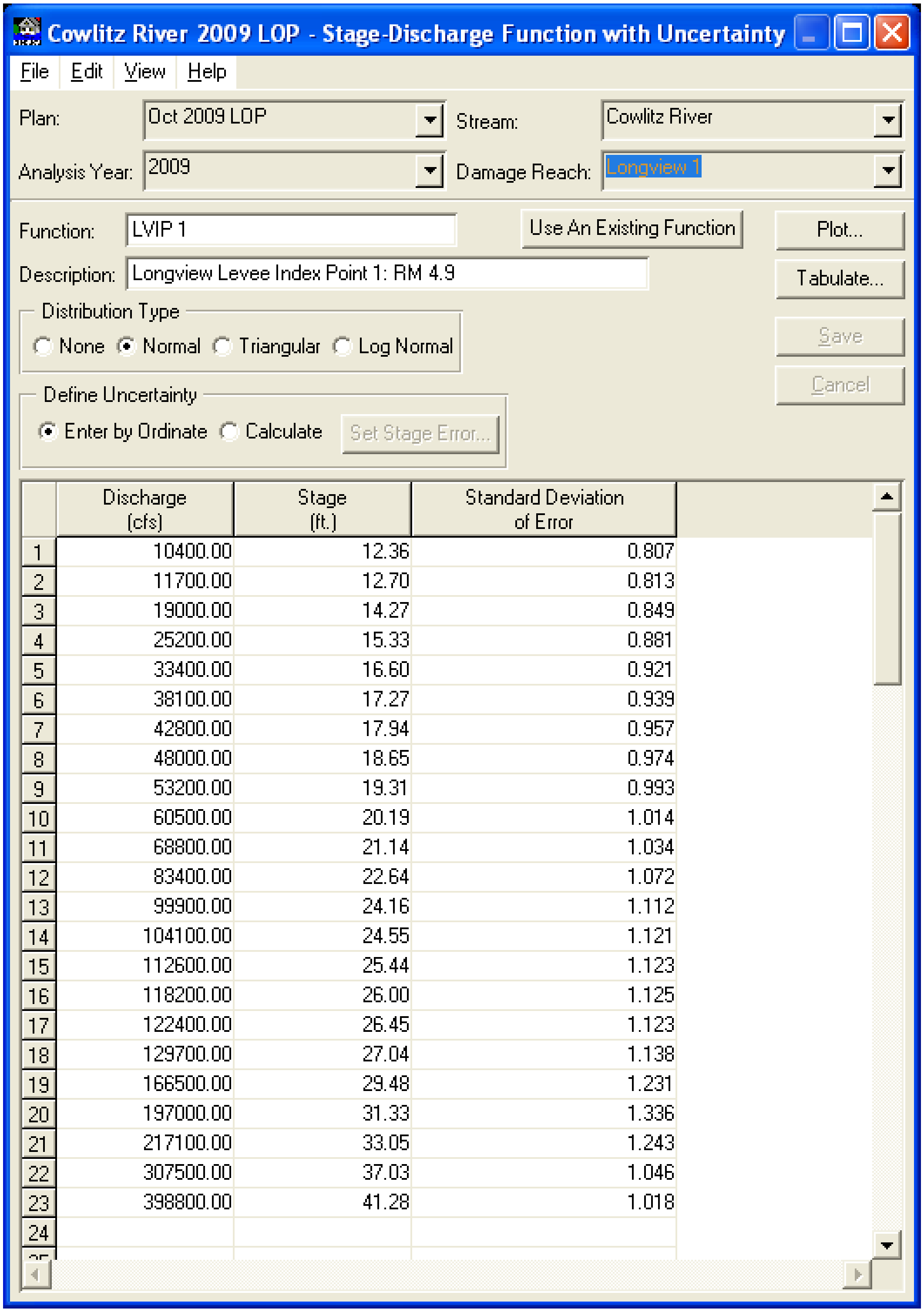
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	10400.00	11.82	0.785
2	11700.00	12.16	0.789
3	19000.00	13.70	0.814
4	25200.00	14.75	0.836
5	33400.00	16.02	0.863
6	38100.00	16.70	0.876
7	42800.00	17.39	0.887
8	48000.00	18.13	0.898
9	53200.00	18.81	0.910
10	60500.00	19.72	0.924
11	68800.00	20.70	0.934
12	83400.00	22.27	0.956
13	99900.00	23.87	0.979
14	104100.00	24.27	0.984
15	112600.00	25.22	0.982
16	118200.00	25.82	0.980
17	122400.00	26.29	0.979
18	129700.00	26.93	0.987
19	166500.00	29.52	1.034
20	197000.00	31.50	1.128
21	217100.00	33.39	1.096
22	307500.00	37.33	0.791
23	398800.00	41.58	0.888
24			

Figure A. 11. Stage Discharge Curve for Kelso 3 (KLIP3)



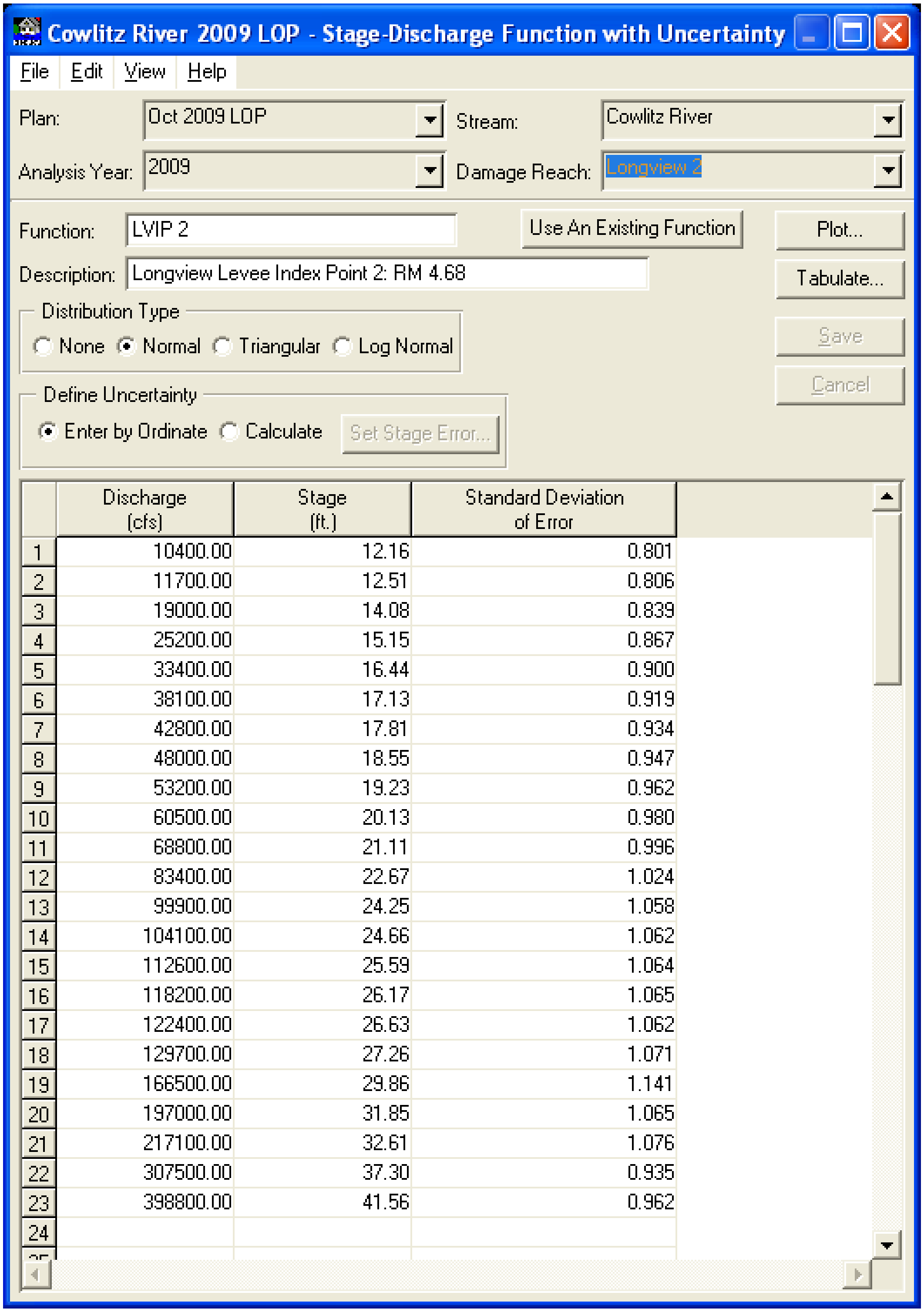
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	10400.00	11.57	0.778
2	11700.00	11.91	0.782
3	19000.00	13.40	0.805
4	25200.00	14.41	0.825
5	33400.00	15.65	0.850
6	38100.00	16.32	0.863
7	42800.00	17.00	0.873
8	48000.00	17.74	0.882
9	53200.00	18.41	0.895
10	60500.00	19.31	0.907
11	68800.00	20.29	0.918
12	83400.00	21.84	0.938
13	99900.00	23.42	0.959
14	104100.00	23.83	0.962
15	112600.00	24.79	0.959
16	118200.00	25.39	0.957
17	122400.00	25.87	0.956
18	129700.00	26.50	0.962
19	166500.00	29.05	1.008
20	197000.00	31.00	1.183
21	217100.00	33.34	1.110
22	307500.00	36.97	0.779
23	398800.00	41.23	0.871
24			

Figure A. 12. Stage Discharge Curve for Kelso 4 (KLIP4)



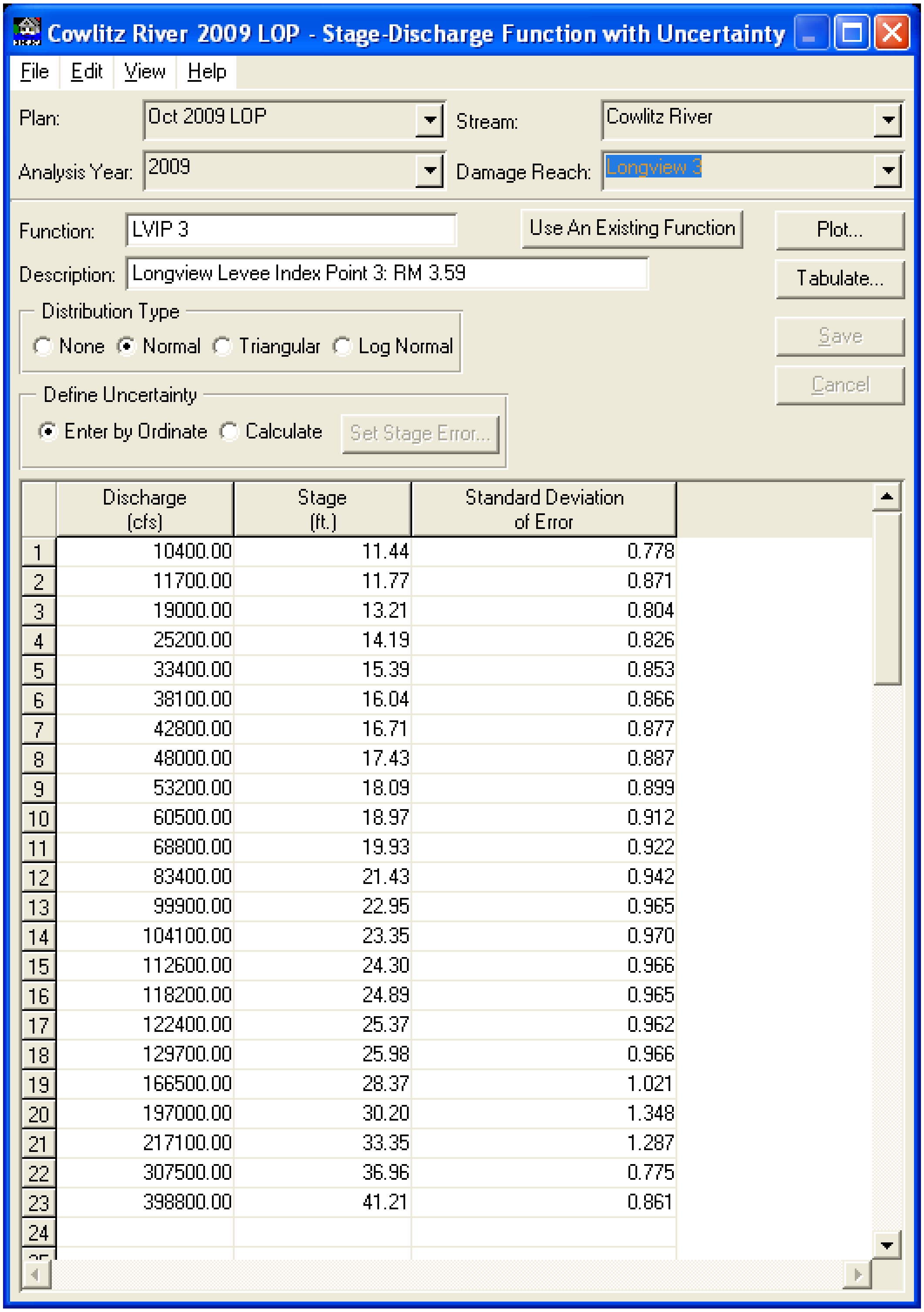
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	10400.00	12.36	0.807
2	11700.00	12.70	0.813
3	19000.00	14.27	0.849
4	25200.00	15.33	0.881
5	33400.00	16.60	0.921
6	38100.00	17.27	0.939
7	42800.00	17.94	0.957
8	48000.00	18.65	0.974
9	53200.00	19.31	0.993
10	60500.00	20.19	1.014
11	68800.00	21.14	1.034
12	83400.00	22.64	1.072
13	99900.00	24.16	1.112
14	104100.00	24.55	1.121
15	112600.00	25.44	1.123
16	118200.00	26.00	1.125
17	122400.00	26.45	1.123
18	129700.00	27.04	1.138
19	166500.00	29.48	1.231
20	197000.00	31.33	1.336
21	217100.00	33.05	1.243
22	307500.00	37.03	1.046
23	398800.00	41.28	1.018
24			

Figure A. 13. Stage Discharge Curve for Longview 1 (LVIP1)



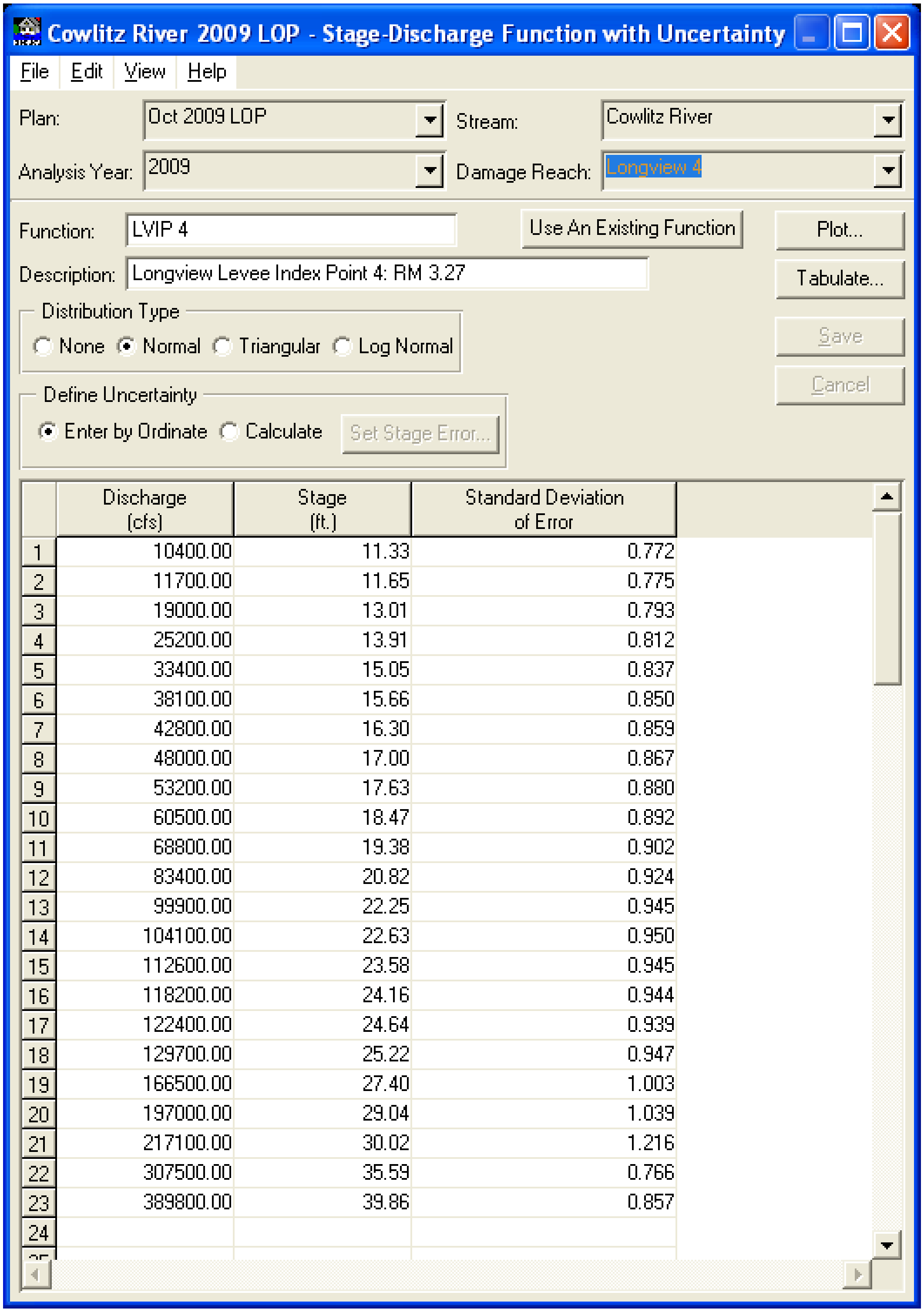
	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	10400.00	12.16	0.801
2	11700.00	12.51	0.806
3	19000.00	14.08	0.839
4	25200.00	15.15	0.867
5	33400.00	16.44	0.900
6	38100.00	17.13	0.919
7	42800.00	17.81	0.934
8	48000.00	18.55	0.947
9	53200.00	19.23	0.962
10	60500.00	20.13	0.980
11	68800.00	21.11	0.996
12	83400.00	22.67	1.024
13	99900.00	24.25	1.058
14	104100.00	24.66	1.062
15	112600.00	25.59	1.064
16	118200.00	26.17	1.065
17	122400.00	26.63	1.062
18	129700.00	27.26	1.071
19	166500.00	29.86	1.141
20	197000.00	31.85	1.065
21	217100.00	32.61	1.076
22	307500.00	37.30	0.935
23	398800.00	41.56	0.962
24			

Figure A. 14. Stage Discharge Curve for Longview 2 (LVIP2)



	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	10400.00	11.44	0.778
2	11700.00	11.77	0.871
3	19000.00	13.21	0.804
4	25200.00	14.19	0.826
5	33400.00	15.39	0.853
6	38100.00	16.04	0.866
7	42800.00	16.71	0.877
8	48000.00	17.43	0.887
9	53200.00	18.09	0.899
10	60500.00	18.97	0.912
11	68800.00	19.93	0.922
12	83400.00	21.43	0.942
13	99900.00	22.95	0.965
14	104100.00	23.35	0.970
15	112600.00	24.30	0.966
16	118200.00	24.89	0.965
17	122400.00	25.37	0.962
18	129700.00	25.98	0.966
19	166500.00	28.37	1.021
20	197000.00	30.20	1.348
21	217100.00	33.35	1.287
22	307500.00	36.96	0.775
23	398800.00	41.21	0.861
24			

Figure A. 15. Stage Discharge Curve for Longview 3 (LVIP3)



	Discharge (cfs)	Stage (ft.)	Standard Deviation of Error
1	10400.00	11.33	0.772
2	11700.00	11.65	0.775
3	19000.00	13.01	0.793
4	25200.00	13.91	0.812
5	33400.00	15.05	0.837
6	38100.00	15.66	0.850
7	42800.00	16.30	0.859
8	48000.00	17.00	0.867
9	53200.00	17.63	0.880
10	60500.00	18.47	0.892
11	68800.00	19.38	0.902
12	83400.00	20.82	0.924
13	99900.00	22.25	0.945
14	104100.00	22.63	0.950
15	112600.00	23.58	0.945
16	118200.00	24.16	0.944
17	122400.00	24.64	0.939
18	129700.00	25.22	0.947
19	166500.00	27.40	1.003
20	197000.00	29.04	1.039
21	217100.00	30.02	1.216
22	307500.00	35.59	0.766
23	389800.00	39.86	0.857
24			

Figure A. 16. Stage Discharge Curve for Longview 4 (LVIP4)

A.3. LEVEE FAILURE CURVES

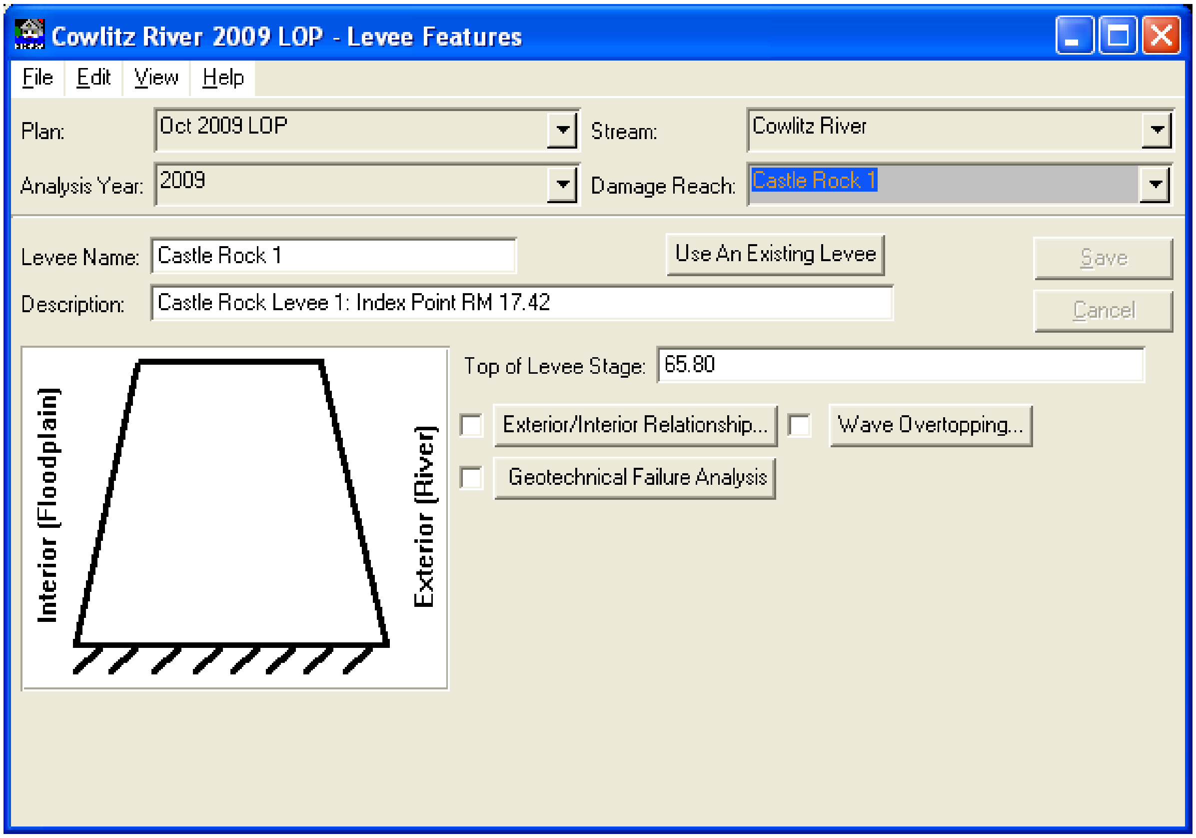
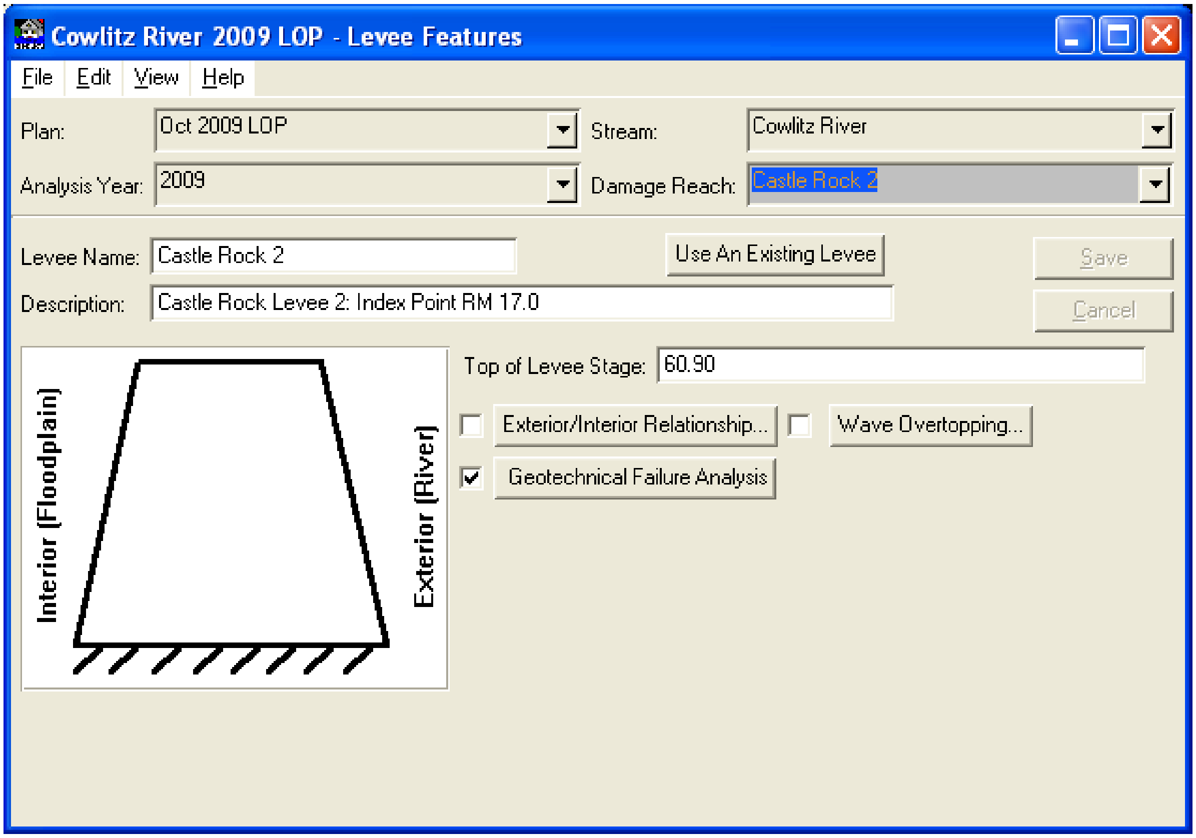


Figure A. 17. Levee Failure Curve for Castle Rock 1 (CRIP1)



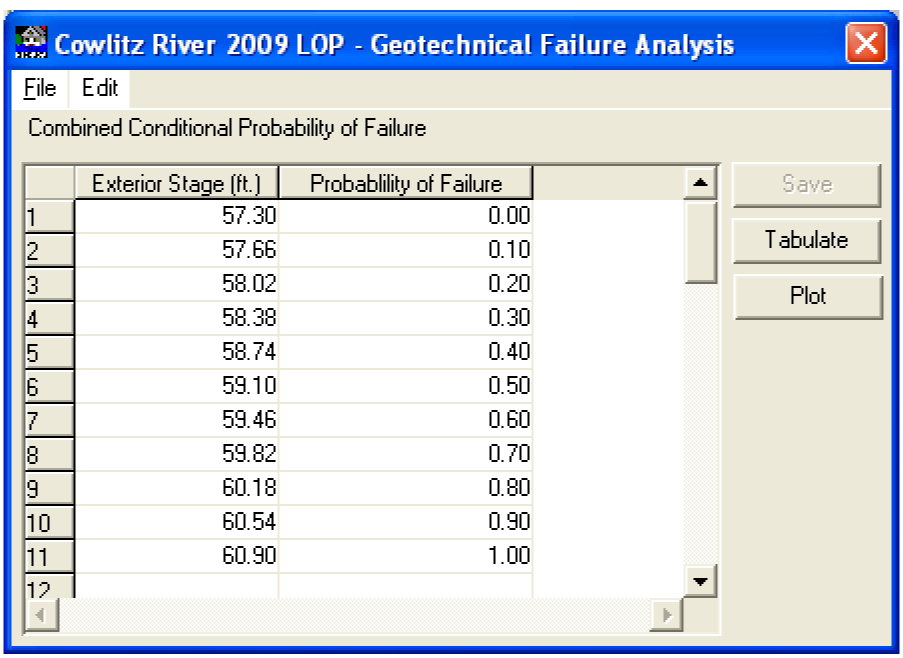


Figure A. 18. Levee Failure Curve for Castle Rock 2 (CRIP2)

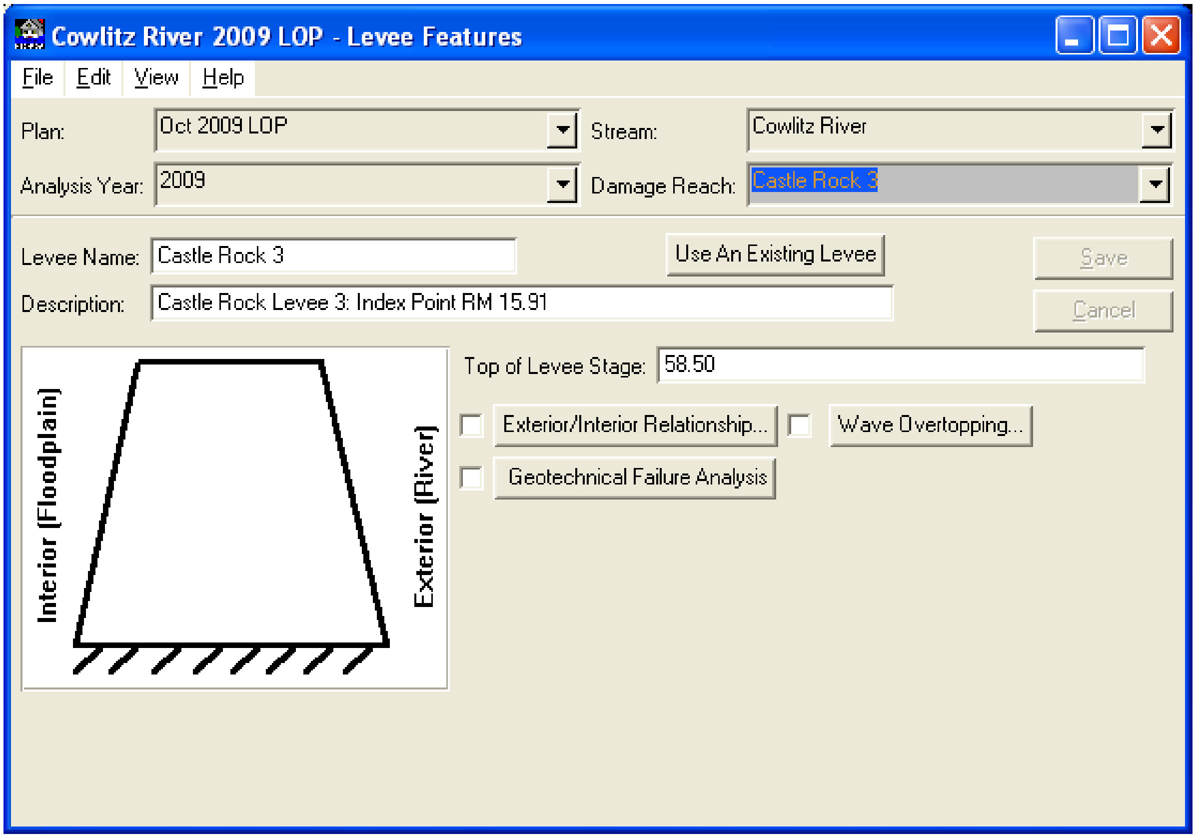


Figure A. 19. Levee Failure Curve for Castle Rock 3 (CRIP3)

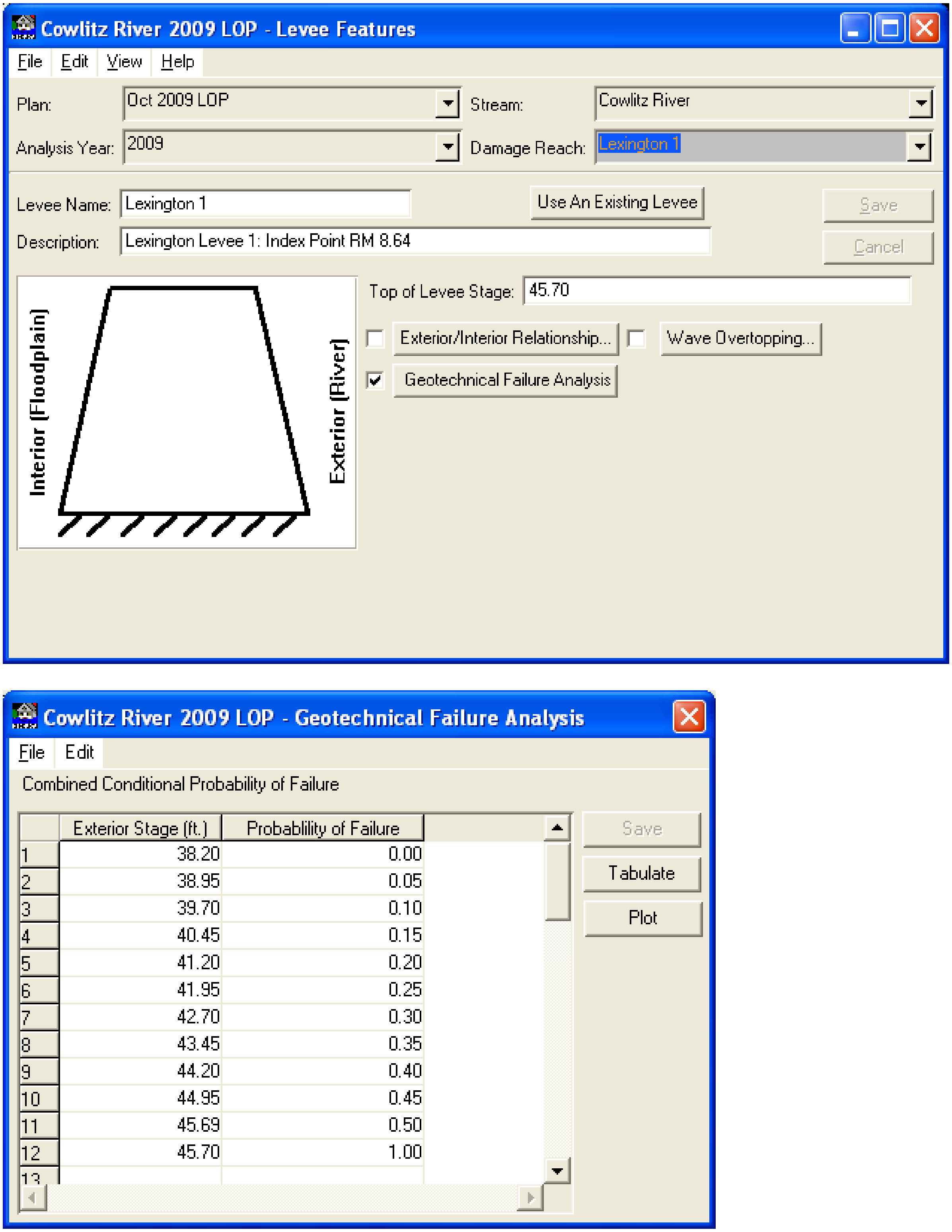


Figure A. 20. Levee Failure Curve for Lexington 1 (LXIP1)

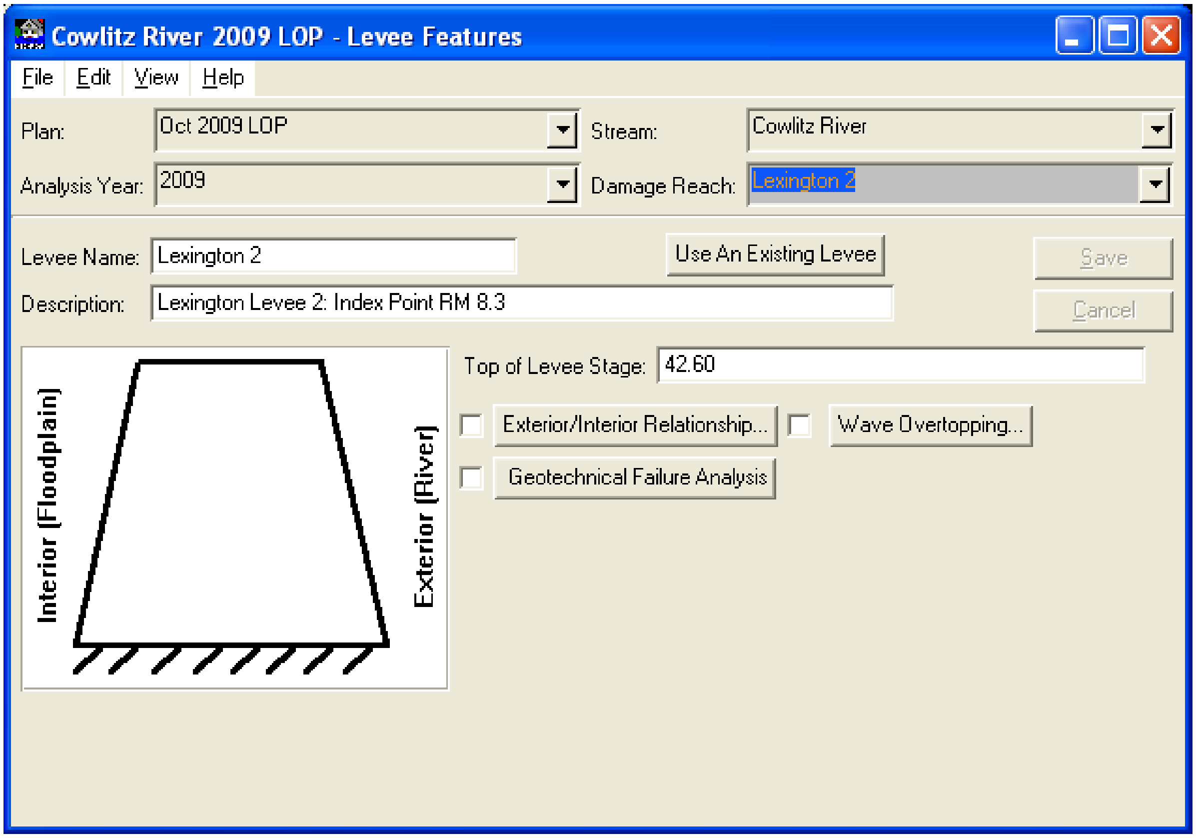


Figure A. 21. Levee Failure Curve for Lexington 2 (LXIP2)

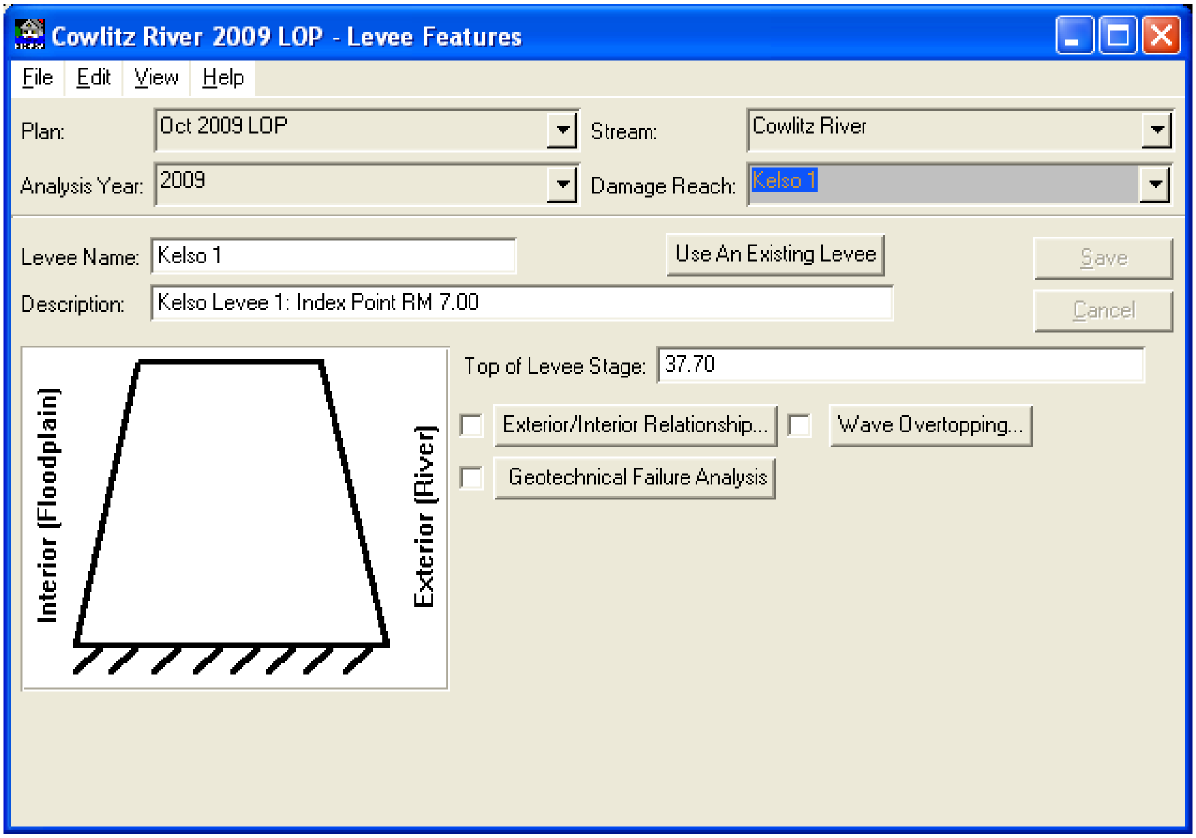
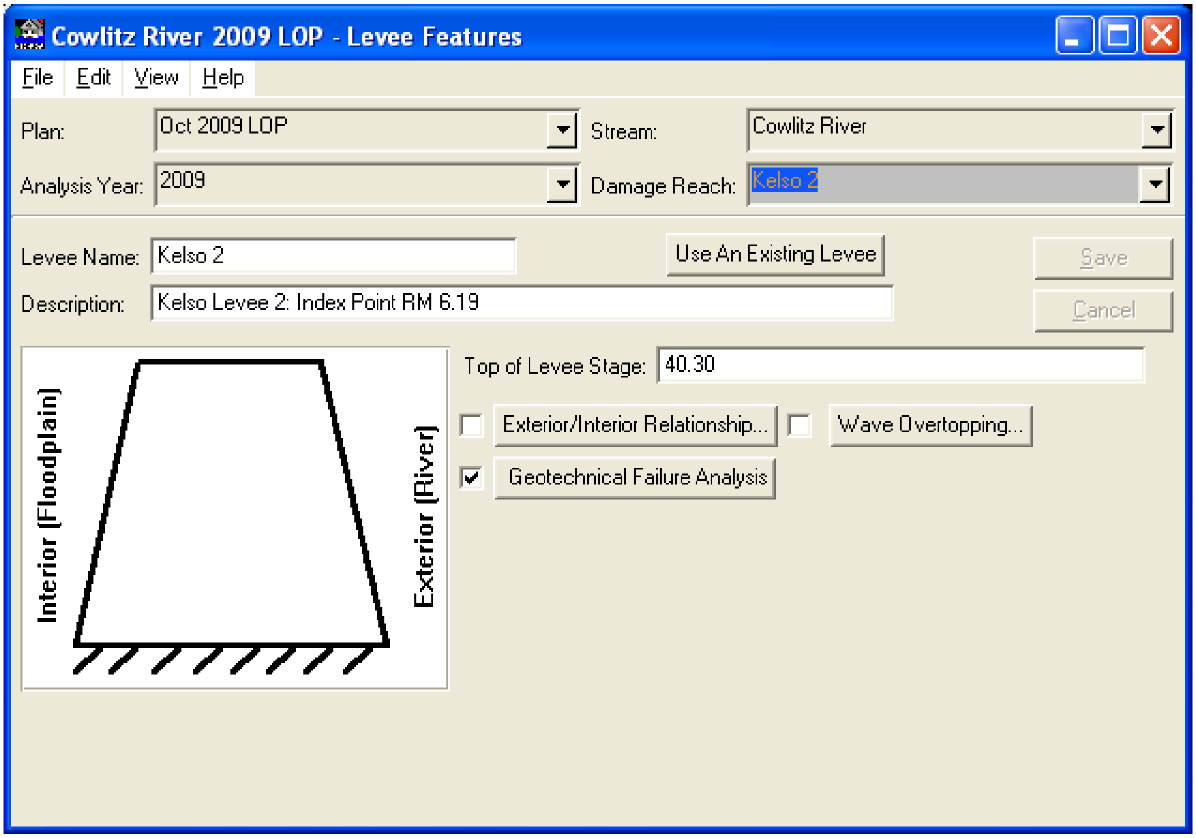
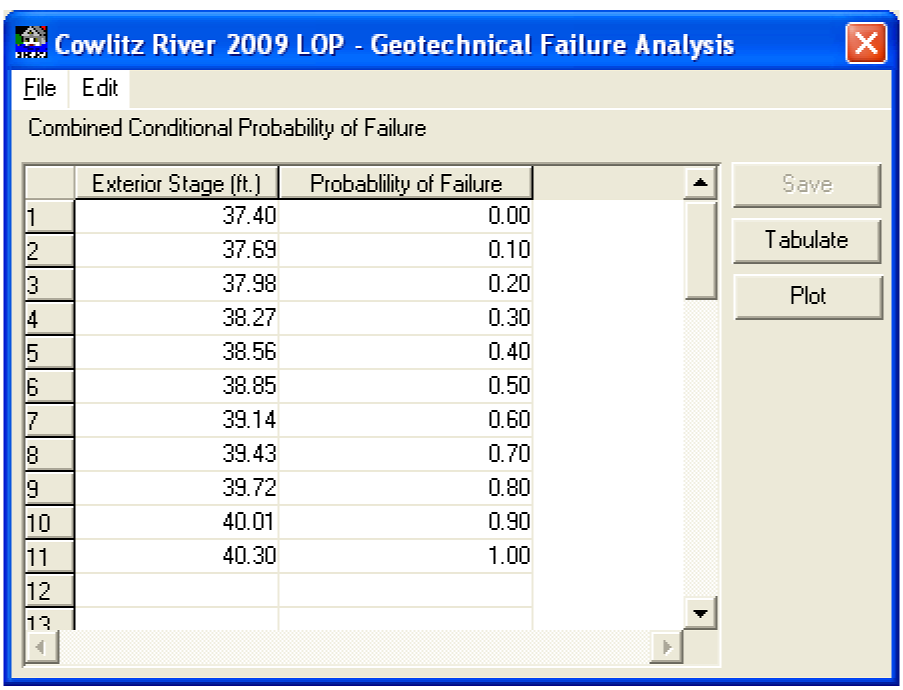


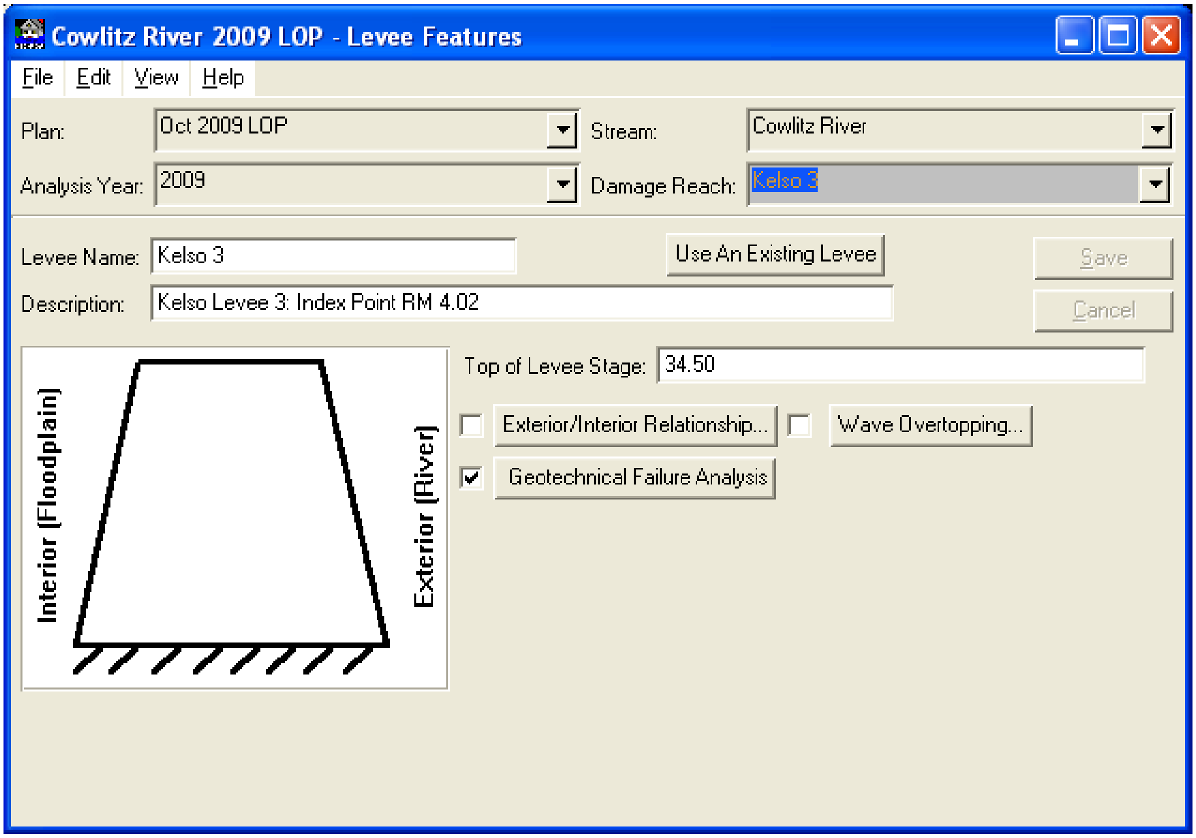
Figure A. 22. Levee Failure Curve for Kelso 1 (KLIP1)





	Exterior Stage (ft.)	Probablility of Failure
1	37.40	0.00
2	37.69	0.10
3	37.98	0.20
4	38.27	0.30
5	38.56	0.40
6	38.85	0.50
7	39.14	0.60
8	39.43	0.70
9	39.72	0.80
10	40.01	0.90
11	40.30	1.00
12		
13		

Figure A. 23. Levee Failure Curve for Kelso 2 (KLIP2)



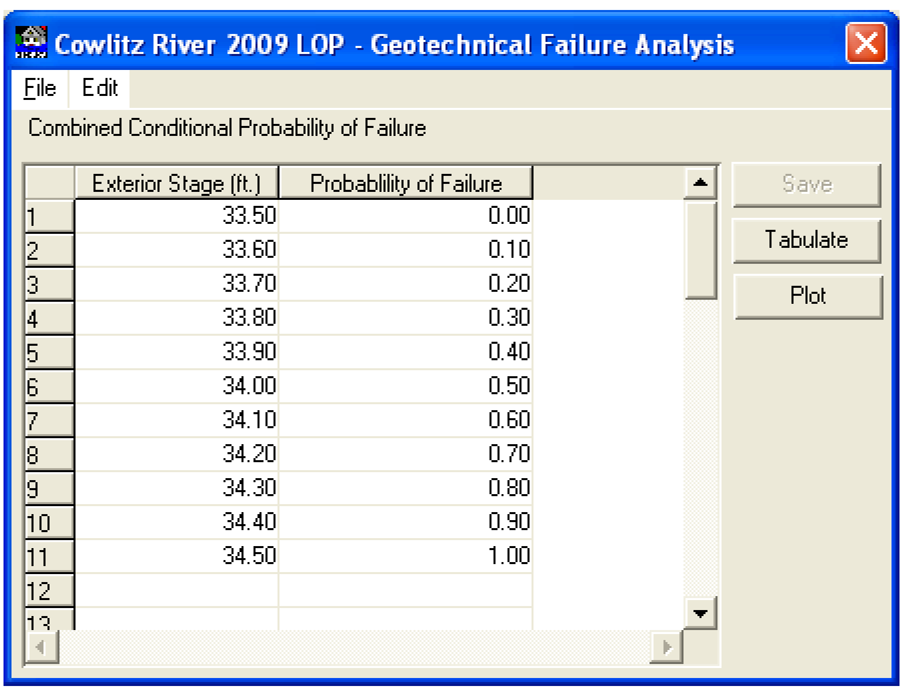
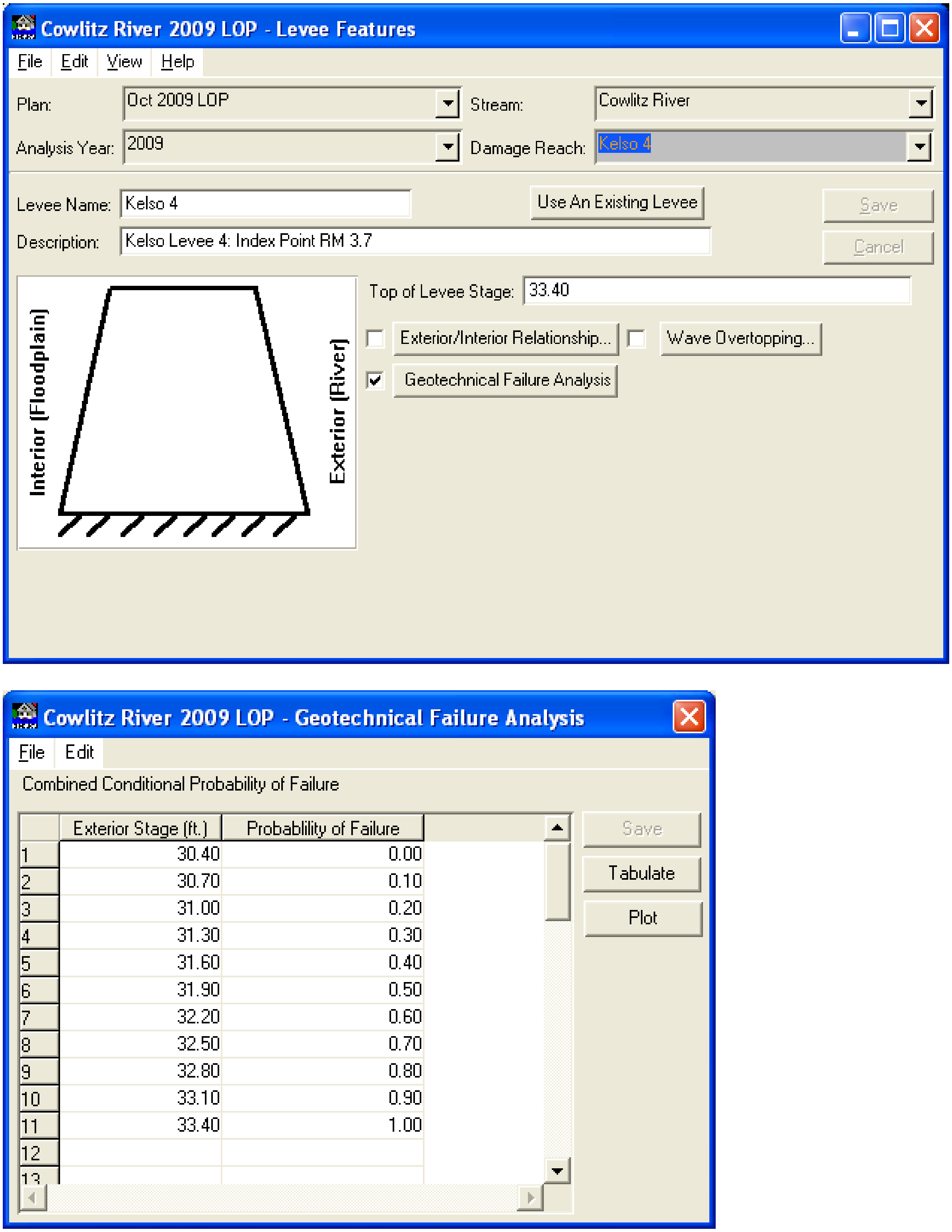


Figure A. 24. Levee Failure Curve for Kelso 3 (KLIP3)



	Exterior Stage (ft.)	Probablility of Failure
1	30.40	0.00
2	30.70	0.10
3	31.00	0.20
4	31.30	0.30
5	31.60	0.40
6	31.90	0.50
7	32.20	0.60
8	32.50	0.70
9	32.80	0.80
10	33.10	0.90
11	33.40	1.00
12		
13		

Figure A. 25. Levee Failure Curve for Kelso 4 (KLIP4)

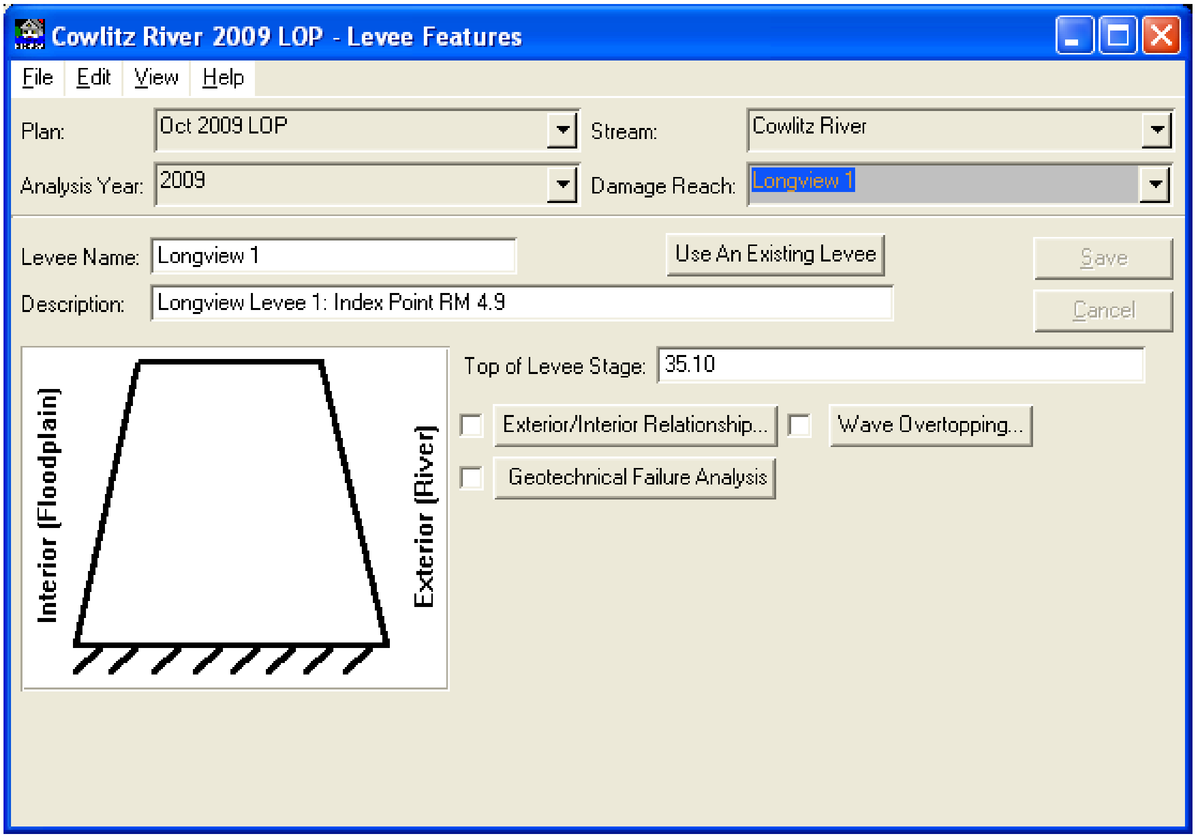
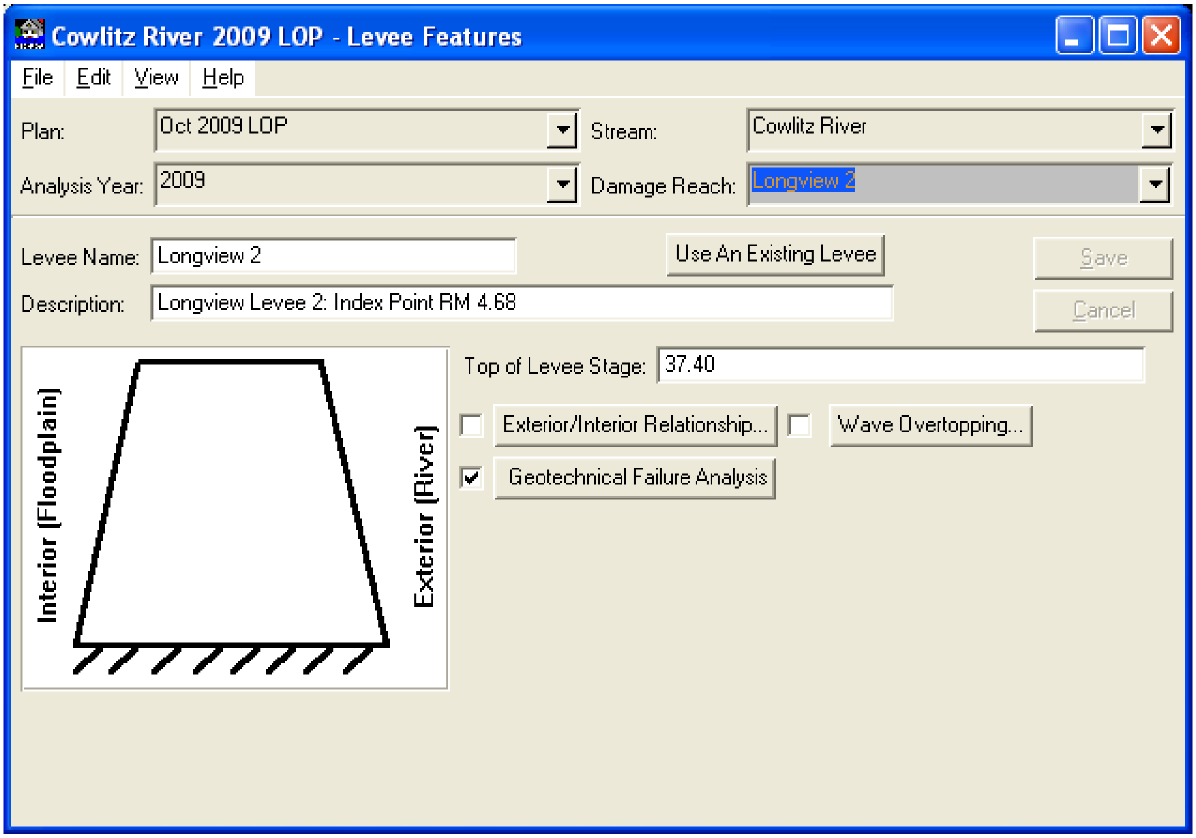
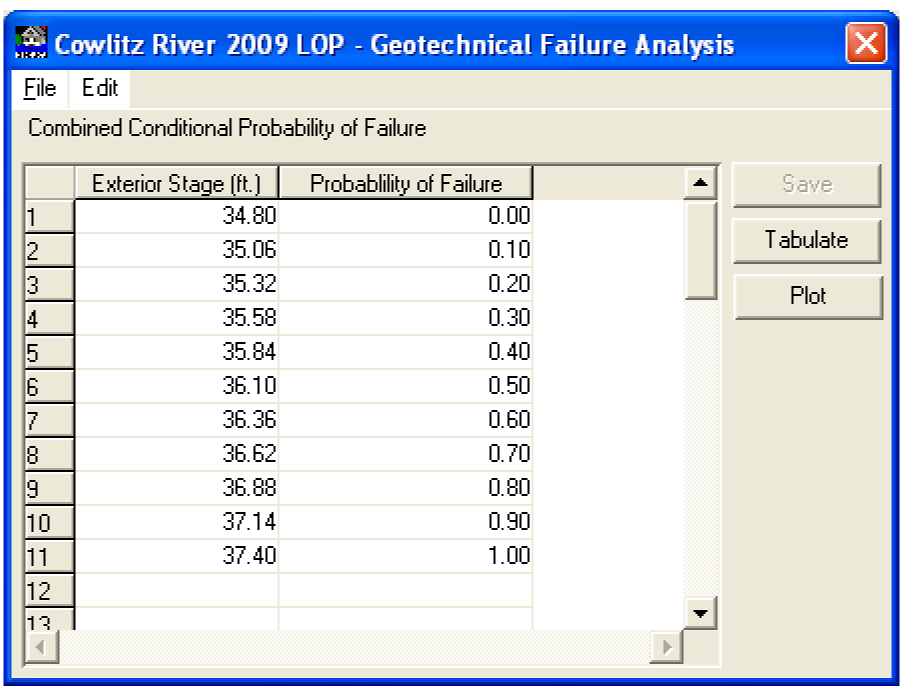


Figure A. 26. Levee Failure Curve for Longview 1 (LVIP1)





	Exterior Stage (ft.)	Probablility of Failure
1	34.80	0.00
2	35.06	0.10
3	35.32	0.20
4	35.58	0.30
5	35.84	0.40
6	36.10	0.50
7	36.36	0.60
8	36.62	0.70
9	36.88	0.80
10	37.14	0.90
11	37.40	1.00
12		
13		

Figure A. 27. Levee Failure Curve for Longview 2 (LVIP2)

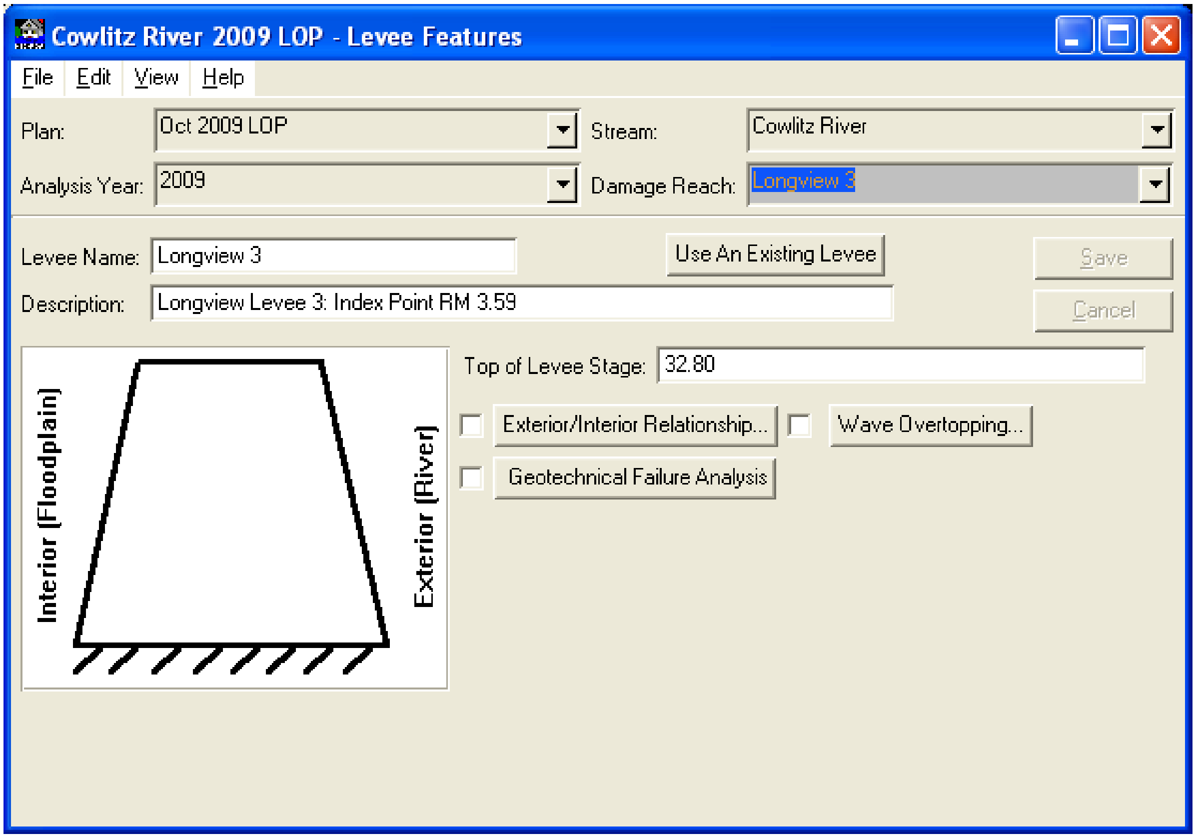


Figure A. 28. Levee Failure Curve for Longview 3 (LVIP3)

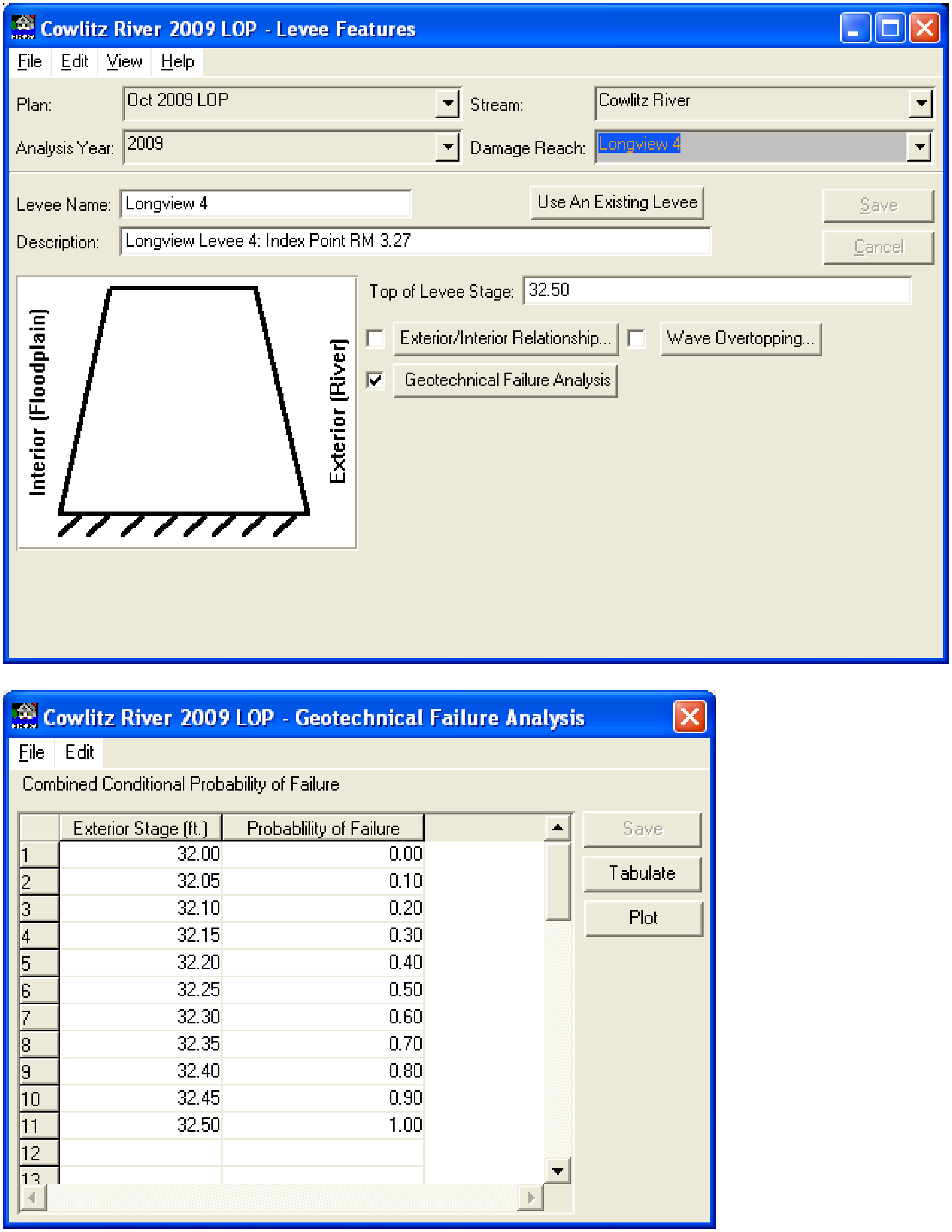


Figure A. 29. Levee Failure Curve for Longview 4 (LVIP4)

A.4. FINAL RESULTS

Project Performance

File Help

Cowlitz River 2009 LOP Project Performance
by Damage Reaches for the Oct 2009 LOP
(Level of Protection October 2009: No Economics) plan for Analysis Year 2009
(Stages in ft.)
Plan was calculated with Uncertainty

Without Project Base Year Performance Target Criteria:
Event Exceedance Probablility = 0.01
Residual Damage = 5.00 %

Stream Name	Stream Description	Damage Reach Name	Damage Reach Description	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
					Median	Expected	10	30	50	10%	4%	2%	1%	.4%	.2%
Cowlitz River	Cowlitz River (F	Longview 4	Longview Levee: 3.27 (300) to 0.67 (40)	levee	0.0007	0.0008	0.0075	0.0185	0.0367	1.0000	1.0000	1.0000	1.0000	1.0000	0.9951
		Longview 3	Longview Levee: 3.59 (330) to 3.27 (300)	levee	0.0008	0.0010	0.0099	0.0245	0.0484	1.0000	1.0000	1.0000	1.0000	0.9998	0.9595
		Kelso 4	Kelso Levee: 3.7 (340) to 1.71 (130)	levee	0.0010	0.0012	0.0122	0.0301	0.0594	1.0000	1.0000	1.0000	1.0000	0.9942	0.8936
		Kelso 3	Kelso Levee: 4.02 (375) to 3.7 (340)	levee	0.0008	0.0009	0.0087	0.0217	0.0429	1.0000	1.0000	1.0000	1.0000	1.0000	0.9778
		Longview 2	Longview Levee: 4.68 (450) to 3.59 (330)	levee	0.0006	0.0006	0.0057	0.0142	0.0282	1.0000	1.0000	1.0000	1.0000	1.0000	0.9993
		Longview 1	Longview Levee: 4.9 (470) to 4.68 (450)	levee	0.0006	0.0007	0.0069	0.0172	0.0341	1.0000	1.0000	1.0000	1.0000	1.0000	0.9953
		Kelso 2	Kelso Levee: 6.19 (600) to 4.02 (375)	levee	0.0007	0.0007	0.0074	0.0184	0.0365	1.0000	1.0000	1.0000	1.0000	1.0000	0.9948
		Kelso 1	Kelso Levee: 7.0 (680) to 6.19 (600)	levee	0.0008	0.0011	0.0109	0.0271	0.0534	1.0000	1.0000	1.0000	1.0000	0.9966	0.9218
		Lexington 2	Lexington Levee: 8.3 (820) to 7.12 (690)	levee	0.0012	0.0015	0.0153	0.0378	0.0741	1.0000	1.0000	1.0000	1.0000	0.9878	0.7992
		Lexington 1	Lexington Levee: 8.64 (860) to 8.3 (820)	levee	0.0019	0.0025	0.0247	0.0606	0.1175	1.0000	0.9976	0.9893	0.9776	0.8852	0.6221
		Castle Rock 3	Castle Rock Levee: 15.91 (1610) to 15.59 (1570)	levee	0.0025	0.0029	0.0282	0.0691	0.1335	1.0000	1.0000	0.9976	0.9908	0.8458	0.3607
		Castle Rock 2	Castle Rock Levee: 17.00 (1710) to 15.91 (1610)	levee	0.0032	0.0045	0.0444	0.1074	0.2033	1.0000	0.9926	0.9671	0.9348	0.6836	0.2592
		Castle Rock 1	Castle Rock Levee: 17.42 (1750) to 17.00 (1710)	levee	0.0010	0.0012	0.0124	0.0307	0.0605	1.0000	1.0000	1.0000	1.0000	0.9968	0.8930

***** - Computations have not been completed.
\+ - Something has changed and computations need to be redone.

Figure A. 30. Results from FDA model for 2009 LOP Estimates

Appendix B. Discharge-Frequency Analysis

B.1. BACKGROUND

Discharge-frequency curves are required to assess the level-of-protection (LOP) at the various levees along the lower ~17 miles of the Cowlitz River from Castle Rock to Kelso/Longview, WA. Since discharges at Castle Rock and below are significantly impacted by flood control and power operations at Mayfield and Mossyrock Dams, regulated discharge-frequency curves are used to assess LOP.

The existing regulated discharge-frequency curve at Castle Rock is from the 1997 Cowlitz River Flood Hazard Study. The methodology used in the 1997 analysis consisted of creating an unregulated discharge-frequency curve using pre-regulation data (1927-1968) and modeled "natural" discharge data from the period of significant regulation (1969-1996), and then using a relationship between regulated and unregulated peak discharges to calculate the regulated discharge-frequency curve. Figure B. 1 shows the regulated-unregulated discharge relationship from the 1997 study and the points used to create the curve (1969 – 1996). Regulated-unregulated pairs for year 1997 to 2009 are also shown in the figure.

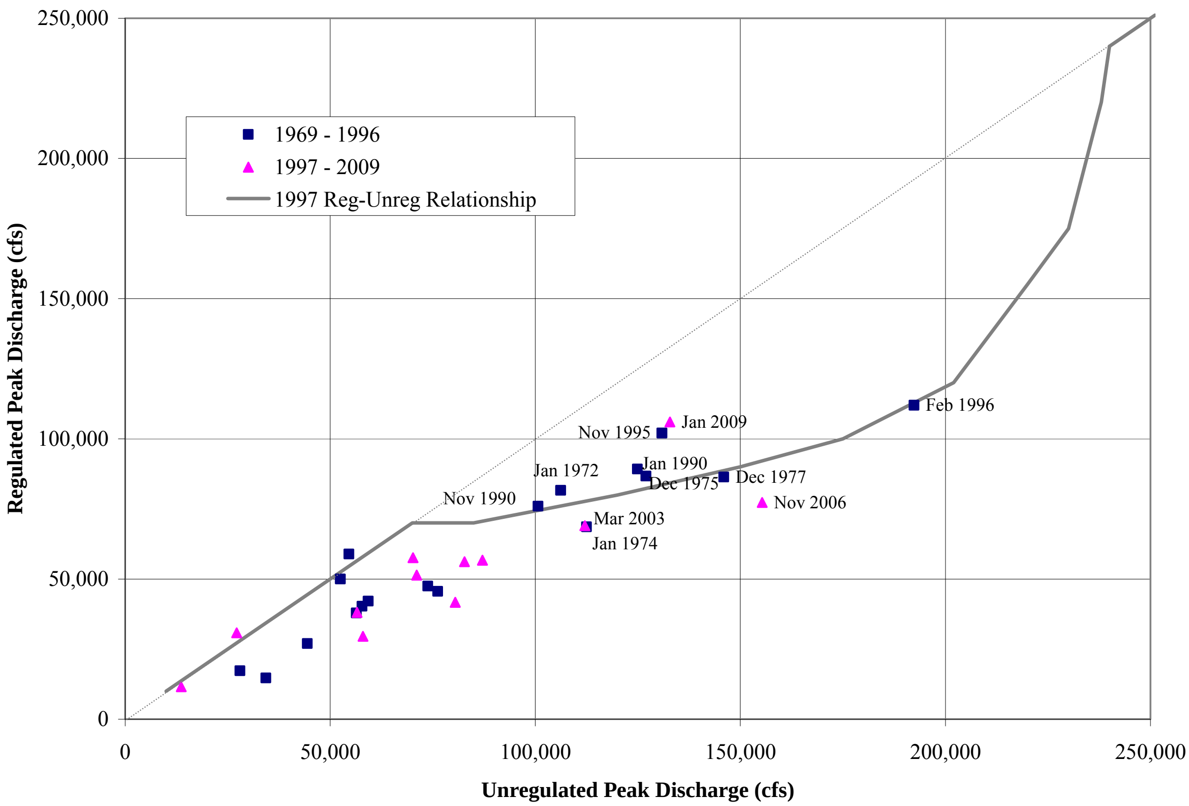


Figure B. 1. Regulated-unregulated discharge relationship used in 1997 study.

A hydrologic analysis was initiated by Portland District (CENWP) in 2008 to update the discharge-frequency curve using a more robust methodology and an updated period-of-record (POR). A draft of

this analysis was presented in the 2008 LOP Update Summary. The finalized hydrologic analysis and discharge-frequency updates are presented in this report.

In the current analysis, the regulated discharge-frequency curve at Castle Rock is graphically fit through a combination of measured and estimated regulated peak discharge data. A synthetic hydrologic analysis is used to supplement the 83 years of measured and estimated data and add resolution to the discharge-frequency relationship for larger events outside of the range of what has occurred in recorded history.

B.2. SYSTEM DESCRIPTION

The Cowlitz River Basin, shown in Figure B. 2, drains the mountainous region between Mount St. Helens and Mt. Rainier to the Columbia River at Longview, WA. The upstream-most levee is at Castle Rock where a USGS gage has been recording discharge data since 1927. Approximately 47% of the area upstream of Castle Rock is above Mossyrock Dam where significant regulation has occurred since April 1968. For reservoir simulation and routing purposes, the area is divided into five tributaries above Castle Rock including local flows. The five locations coincide with the major, long-term USGS gages in the basin.

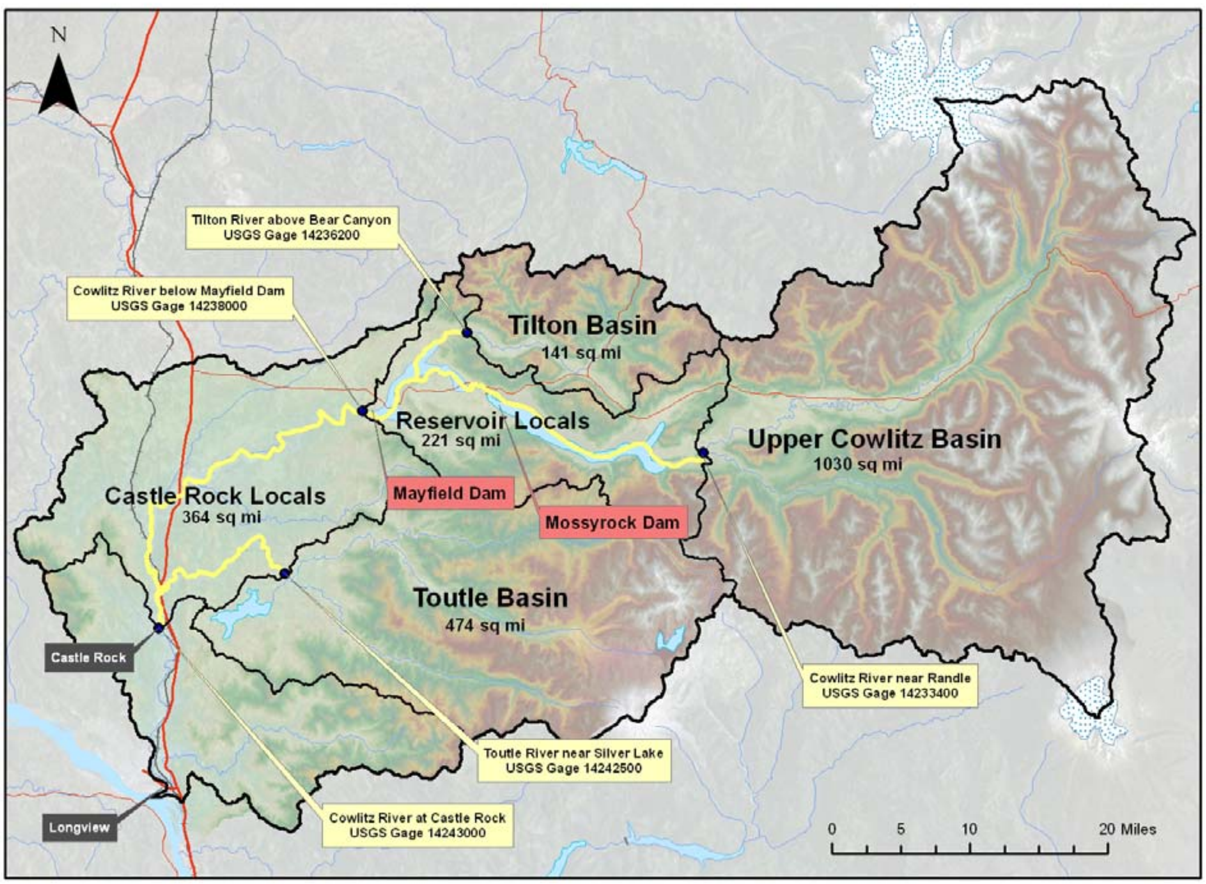


Figure B. 2. Cowlitz basin map including the dams, tributaries with drainage areas, ResSim river reaches, and key USGS gages.

A HEC-ResSim routing and reservoir simulation model of the Cowlitz Basin has been in use for several years by CENWP. Its primary function has been to estimate unregulated peak discharges at Castle Rock. In the current study, the HEC-ResSim model is used to simulate reservoir operations and route hypothetical tributary flows down to Castle Rock as part of the synthetic analysis. The ResSim river and reservoir network is shown in Figure B. 3.

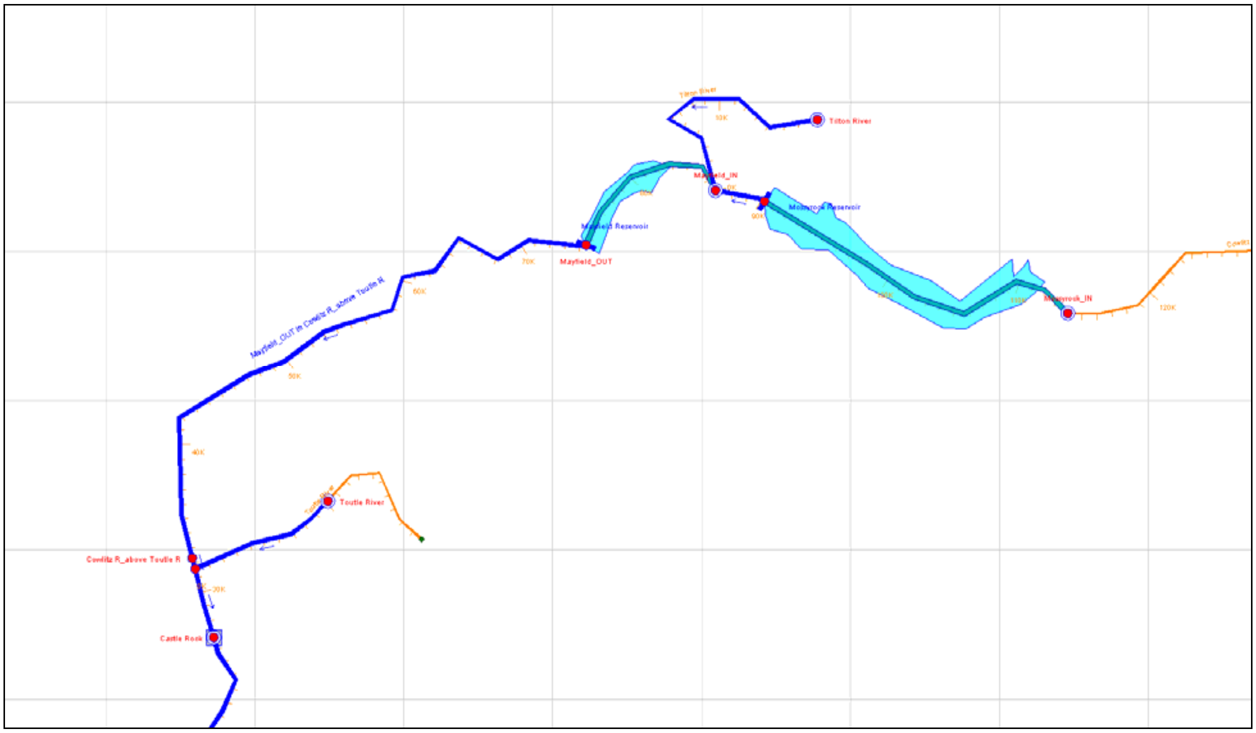


Figure B. 3. HEC-ResSim model of the Cowlitz Basin.

B.3. UNREGULATED AND REGULATED DISCHARGE DATA

B.3.1. Measured Data

The Cowlitz River at Castle Rock, WA gage (USGS 14243000) has been in operation since December 1926. There is a systematic record length of 83 years, Water Years (WY) 1927-2009, available for the discharge-frequency analysis. Maximum annual instantaneous peak discharge data and average daily discharge are available on the USGS website. Hourly data are available for recent years on the USGS's instantaneous data archive website. Additional short-interval data have been provided by the USGS upon special request for the current and past studies.

The discharge data from the 83-year POR at Castle Rock are not homogeneous; stream flow has been significantly impacted since 1969 due to reservoir regulation in Riffe Lake (Mossyrock Dam) and Mayfield Lake. The measured Castle Rock data can be divided into two homogeneous periods: natural (WYs 1927-1968) and regulated (WYs 1969-2009).

Substantial effort has been put into estimating unregulated data during the regulated period, and vice versa, in order to maximize POR for a particular dataset. Additional long-term USGS gages in the Cowlitz basin are used in the analysis for simulating flood events and estimating regulated and unregulated discharges. Figure B. 4 shows the gages used in the analysis.

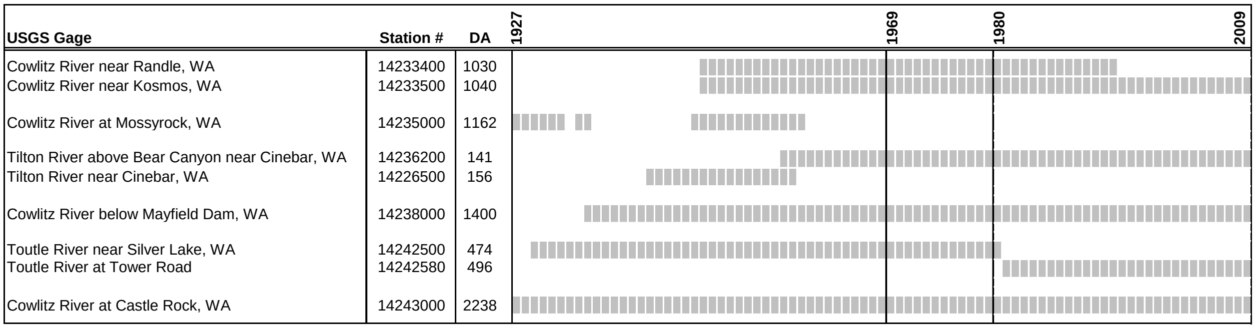


Figure B. 4. USGS gages in the Cowlitz Basin used in the Castle Rock discharge-frequency analyses.

B.3.2. Estimated Unregulated Data

For both the 1997 analysis and the present analysis, “natural” or unregulated discharge data are necessary for creating unregulated discharge-frequency curves and for assessing the relationship between regulated and unregulated discharges. In both analyses, reservoir storage data and discharge data from throughout the basin are used to simulate without-project conditions and calculate unimpaired hydrographs at Castle Rock for the largest storms of each year post-1968. From these hydrographs, unregulated peak and volume data can be calculated. Short interval data are needed to accurately calculate local flows between gaged locations and changes in storage in the reservoir. In the present analysis, HEC-ResSim is used to route hydrographs and simulate no-regulation conditions.

Since the 1997 report, there have been two noteworthy events: the November 2006 and the January 2009 events. The November 2006 flood event was a very large event concentrated in the Tilton and Upper Cowlitz basins. The storm produced an unregulated peak discharge of 155,000 cfs at Castle Rock, which is the third largest in the POR. At the beginning of the event, Riffe Lake (Mossyrock) was well below the rule curve at elevation 737 ft with 439,500 acre-feet of storage available for flood control. The scheduled amount of storage would have been 200,000 acre-feet. The extra flood storage capacity and the fact that the storm was concentrated above the reservoirs resulted in a drastic reduction in peak discharge at Castle Rock to just 77,300 cfs.

The flood in early January 2009 was very different than the November 2006 flood. The unregulated peak discharge was estimated at 133,000 cfs, which is the 5th largest in the POR. Prior to the event, the elevation of Riffe Lake was 712.5 feet on January 6, with available storage of 665,500 acre-ft, almost twice the maximum required storage of 360,000 acre-ft, and more than 200,000 acre-ft more than what was available in November 2006. The regulated peak discharge measured at Castle Rock on January 8, 2009 was 106,000 cfs, which is the second highest regulated discharge recorded since regulation began in 1969. The small degree of flood peak reduction (about 20%) in this case is attributed to the fact that the storm was largely concentrated below the reservoirs.

Figure B. 5 shows the maximum annual 1-day unregulated discharges for the POR at Castle Rock. The unregulated discharges for years with significant regulation (1969 and later) in this figure are calculated using a basic routing model with daily data.

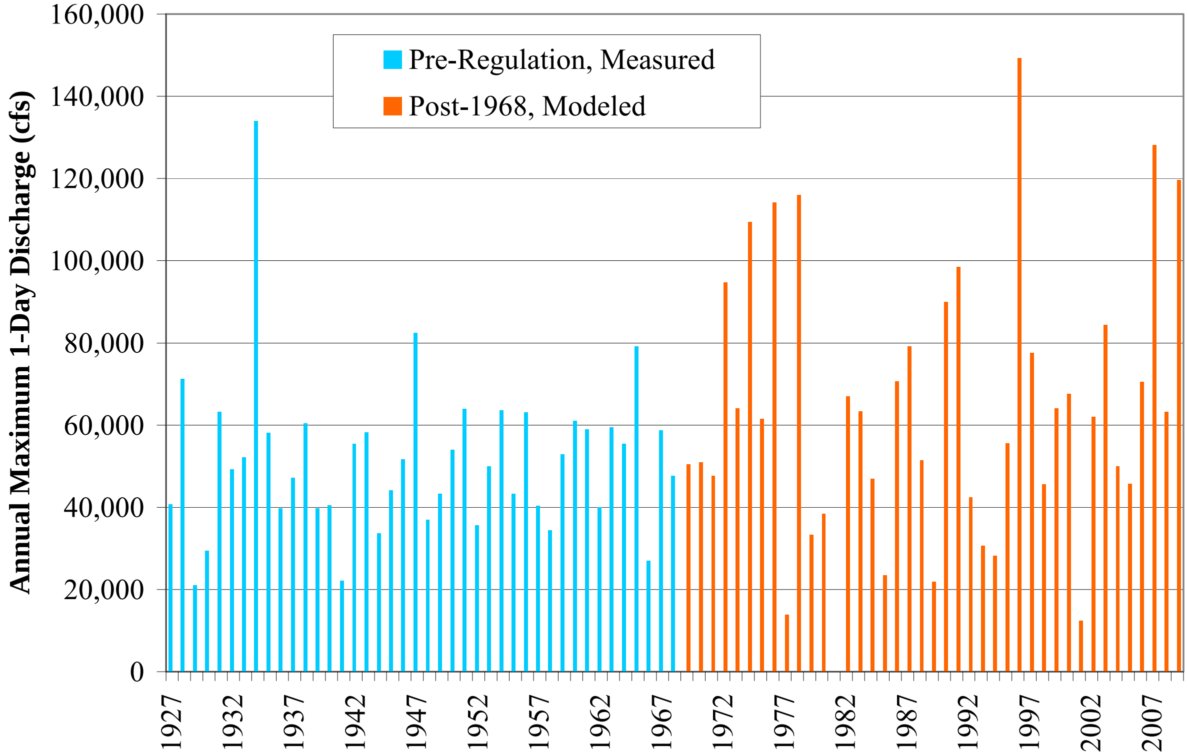


Figure B. 5. Maximum annual 1-day unregulated discharges for Cowlitz River at Castle Rock.

B.3.3. Estimated Regulated Data

Due to a lack of short interval data and intermittent gage records, performing a detailed simulation of all the pre-regulation years is not feasible. The approach taken in this study is to simulate only the events/WYs that would impact the upper end of the regulated frequency curve where LOP is estimated, and to use a more generic, less accurate approach for the less impactful WYs.

To identify the potentially influential WYs, the annual maximum 1-day discharges from 1927 to 1968 are combined with the available calculated unregulated 1-day discharges from WYs 1969 to the present, and are ranked from largest to smallest in the Table B. 1. The table includes the November 1995 event and the 2009 peak discharge calculated in the present study.

Table B. 1. Regulated and Unregulated Discharges at Castle Rock from WYs 1927 to 2009.

Rank	WY	Unregulated 1-Day Flow (cfs)	Regulated Peak Flow (cfs)		Rank	WY	Unregulated 1-Day Flow (cfs)	Regulated Peak Flow (cfs)
1	1996	149,354	112,000		43	1959	52,900	???
2	1934	134,000	???		44	1933	52,200	???
3	2007	128,176	77,300		45	1946	51,700	???
4	Nov-95	124,297	105,000		46	1988	51,371	42,100
5	2009	119,656	106,000		47	1970	50,924	37,900
6	1978	115,990	86,400		48	1969	50,449	40,300
7	1976	114,186	86,700		49	1953	50,000	???
8	1974	109,477	68,600		50	2004	49,963	38,100
9	1991	98,503	76,000		51	1932	49,200	???
10	1972	94,699	81,600		52	1971	47,718	50,000
11	1990	89,995	89,200		53	1968	47,600	???
12	2003	84,391	69,000		54	1937	47,200	???
13	1947	82,400	???		55	1984	46,993	49,600
14	1987	79,128	77,600		56	2005	45,707	29,600
15	1965	79,100	???		57	1998	45,631	30,800
16	1997	77,637	56,700		58	1945	44,200	???
17	1928	71,300	???		59	1949	43,300	???
18	1986	70,690	62,700		60	1955	43,300	???
19	2006	70,563	56,200		61	1992	42,460	27,000
20	2000	67,621	41,700		62	1927	40,800	???
21	1982	67,034	65,500		63	1940	40,500	???
22	1999	64,104	57,600		64	1957	40,400	???
23	1973	64,102	45,600		65	1962	40,100	???
24	1951	63,900	???		66	1936	39,800	???
25	1954	63,600	???		67	1939	39,800	???
26	1983	63,322	67,300		68	1980	38,433	33,400
27	1931	63,200	???		69	1948	37,000	???
28	2008	63156	64,300		70	1952	35,600	???
29	1956	63,100	???		71	1958	34,500	???
30	2002	62,084	51,400		72	1944	33,700	???
31	1975	61,567	58,900		73	1979	33,380	39,400
32	1960	61,000	???		74	1993	30,676	14,700
33	1938	60,500	???		75	1930	29,500	???
34	1963	59,500	???		76	1994	28,283	17,300
35	1961	59,000	???		77	1966	27,000	???
36	1967	58,800	???		78	1985	23,444	23,600
37	1943	58,300	???		79	1941	22,100	???
38	1935	58,100	???		80	1989	21,970	23,600
39	1995	55,575	47,500		81	1929	21,100	???
40	1942	55,400	???		82	1977	13,929	15,000
41	1964	55,400	???		83	2001	12,490	11,600
42	1950	54,000	???					

Of this data set, only four storms from 1927 to 1968 produced unregulated discharges that would have likely enacted preventative flood regulation, i.e. those flows projected to be larger than 70,000 cfs at

Castle Rock. Of these events, only the December 1933 event (WY 1934) was above 90,000 cfs, the low end of the major flood damage level. The next three largest events are the 1947, 1965, and 1928 events, ranking 13, 15, and 17, respectively.

Because this study is primarily concerned with the magnitude and frequency of large events that will affect estimates of the LOP, which are between the 0.01 and 0.005 AEP discharges, a detailed simulation was performed only for the December 1933 event. For all other pre-regulated years, regulated peak discharges were estimated using a range of regulated-unregulated discharge relationships.

B.3.3.1. The December 1933 Event

The December 1933 flood event was comprised of two large flood waves spaced just 13- to 14-days peak-to-peak. It produced by far the largest 15-day average discharge at Castle Rock in the POR, averaging just greater than the bankfull discharge, which defines the primary flow objective for Mossyrock Dam. The first flood wave had a 1-day average discharge of 102,000 cfs and a 4-day average discharge of 85,500 cfs, the two of which would have ranked 7th and 3rd largest on record were they not followed by the larger flood wave. The second flood wave had the second largest 1- through 4-day average discharges, and it had the largest 5- and 7-day runoff volumes on record, averaging 106,000 cfs and 96,000 cfs, respectively. The peak discharge of 139,000 cfs is the largest flow measured at Castle Rock, and the 4th largest when including all of the estimated unregulated peaks that have occurred since regulation began in 1969.

The Mossyrock and Mayfield Dams Water Control Manual specify that the dams were designed to contain all floods of record including this, the largest recorded event at the time. Following the water control manual, the regulated discharge at Castle Rock would be kept at or below bankfull flow by utilizing all of the flood storage capacity. The design only looks at the second, larger wave and assumes the reservoir is at the rule curve with 100% available flood storage capacity at the beginning of the event.

A review of operating procedures during historical events revealed that Mossyrock Dam seldom aggressively evacuates storage immediately following an event; more commonly, water stored above rule curve is released at a steady rate between 15,000 and 25,000 cfs, sometimes at the powerhouse discharge of 10,000 cfs. The largest spill in the history of Mossyrock Dam was 32,000 cfs during the November 1995 flood event. It should also be noted that the USGS flood stage below Mayfield Dam occurs at 25,000 cfs, and that notifications to local authorities are required when releases will exceed 20,000 cfs.

The December 1933 event was modeled to determine what the peak regulated discharge could have been under more realistic operating conditions. Hourly inflow hydrographs were created using daily and instantaneous peak discharge data for Cowlitz River at Mossyrock and Castle Rock. Various instantaneous peak discharge and time-of-peak data provided by the USGS were also used in shaping the hydrographs. The hydrographs were created by passing through observed peaks and balancing average daily flows. Hourly hydrographs for locals were calculated as the difference between the routed upstream hydrograph at Mossyrock and Castle Rock. Figure B. 6 shows three hydrographs and the data used to create them.

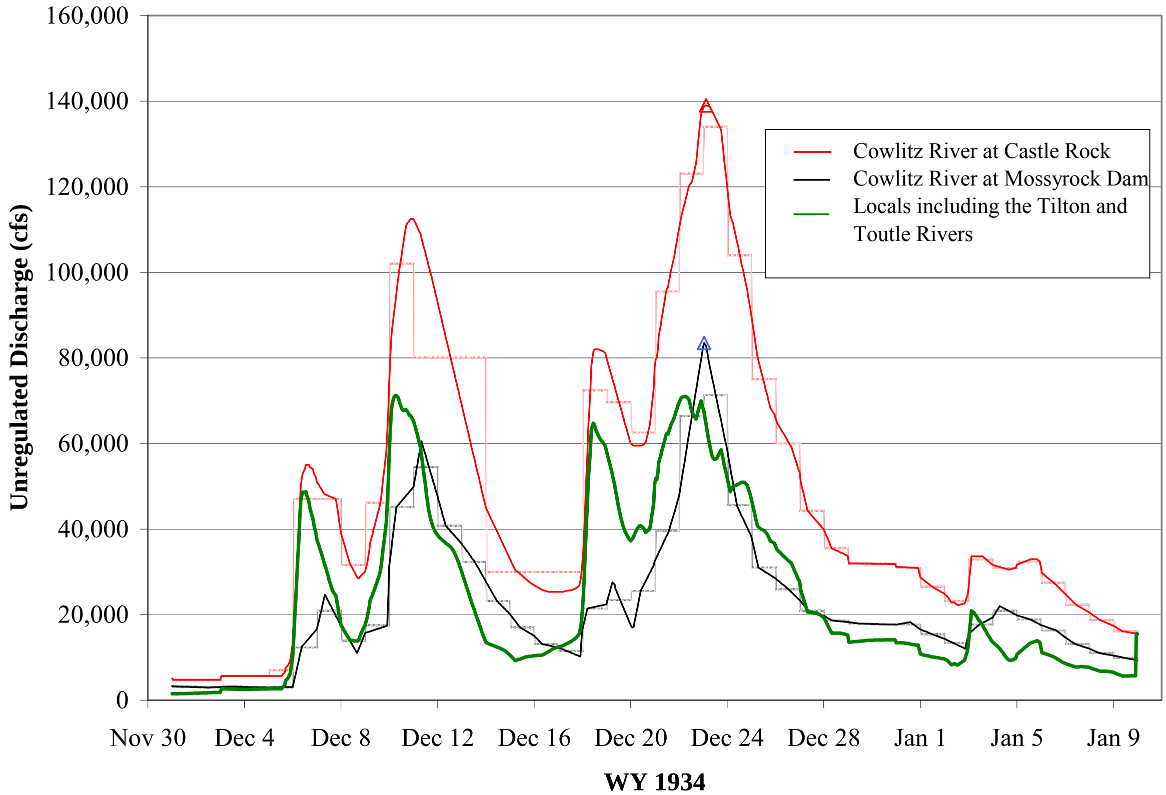


Figure B. 6. Synthetic hourly hydrographs passing through instantaneous peaks and matching daily average discharges were used to simulate the December 1933 flood event.

An evacuation rate of 25,000 cfs was assumed for the period between the two flood waves. This is a fairly aggressive rate but is assumed to be a conservative estimate of what the maximum discharge could have been. Ideal and less-than-ideal release operation schemes were assumed for early release during the second, larger flood wave to assess the sensitivity of regulation on the downstream regulated peak. From these different scenarios, a range of regulated peak discharges at Castle Rock were calculated.

Using an evacuation rate of 25,000 cfs brings the pool back down to an elevation of 759 feet, about 62% of full storage capacity available, by the time the second wave hits. Without any preventative regulation, the reservoir could have filled prior to the inflow to Mossyrock peaking, resulting in a downstream peak of 130,000 cfs. This estimate, however, is assumed to be overly conservative. Assuming some preventative regulation measures, i.e. if the regulators had decent forecasts and initiated early releases, the regulated peak discharge at Castle Rock would likely have ranged from about 100,000 to 120,000 cfs. A more precise estimate, if possible, is not necessary for this analysis.

Below are two figures (Figure B. 7and Figure B. 8) showing ResSim results for a scenario with moderate to poor regulation, showing a late ramp-up to 50,000 cfs as the pool approaches full, and the resulting fill-and-spill occurring approximately 12 hours after the inflow peak. This scenario resulted in a regulated peak at Castle Rock of 118,000 cfs.

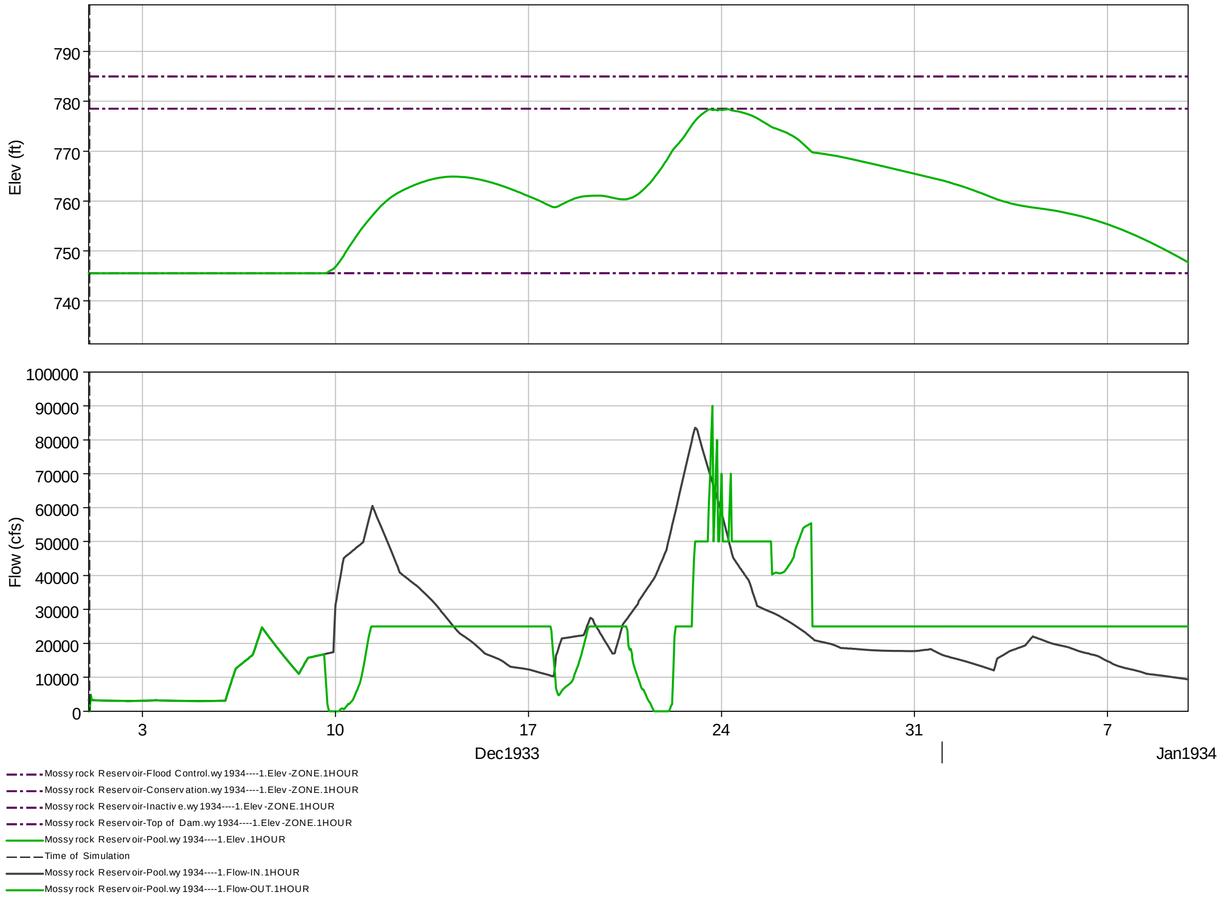


Figure B. 7. Mossyrock pool elevation (top graph) and the inflow and outflow (bottom graph) for a scenario. Note, the presence of oscillations in dam outflow as the reservoir reaches full and begins to pass inflow is due to a minor instability in the model, but it has little to no impact on the downstream flows as the drastic flow changes are attenuated into a smooth hydrograph in the reach below Mayfield Dam.

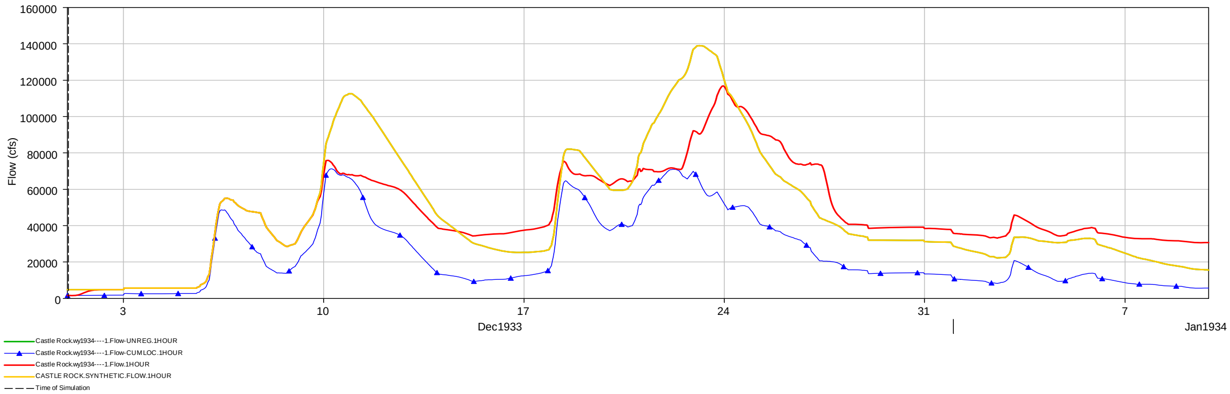


Figure B. 8. The regulated discharge at Castle Rock for this scenario, shown in Red, peaks at 118,000 cfs, and is caused by Mossyrock Dam releasing inflows as the flood storage capacity in Riffe Lake is diminished. The unregulated and cumulative local flows are shown in yellow and blue, respectively.

B.3.3.2. All Other Pre-Regulation Water Years

Figure B. 9 shows the regulated peak discharges plotted against their corresponding unregulated 1-day discharges for all regulated WY's with the exception of 1981. The green lines represent pre-regulation annual maximum 1-day discharges. For unregulated 1-day discharges less than 90,000 cfs, the relationships m = 1.03, m = 0.85, and m = 0.67 represent maximum, average, and minimum relationships, respectively. These relationships are used for estimating a range of regulated discharges from the observed, unregulated discharges.

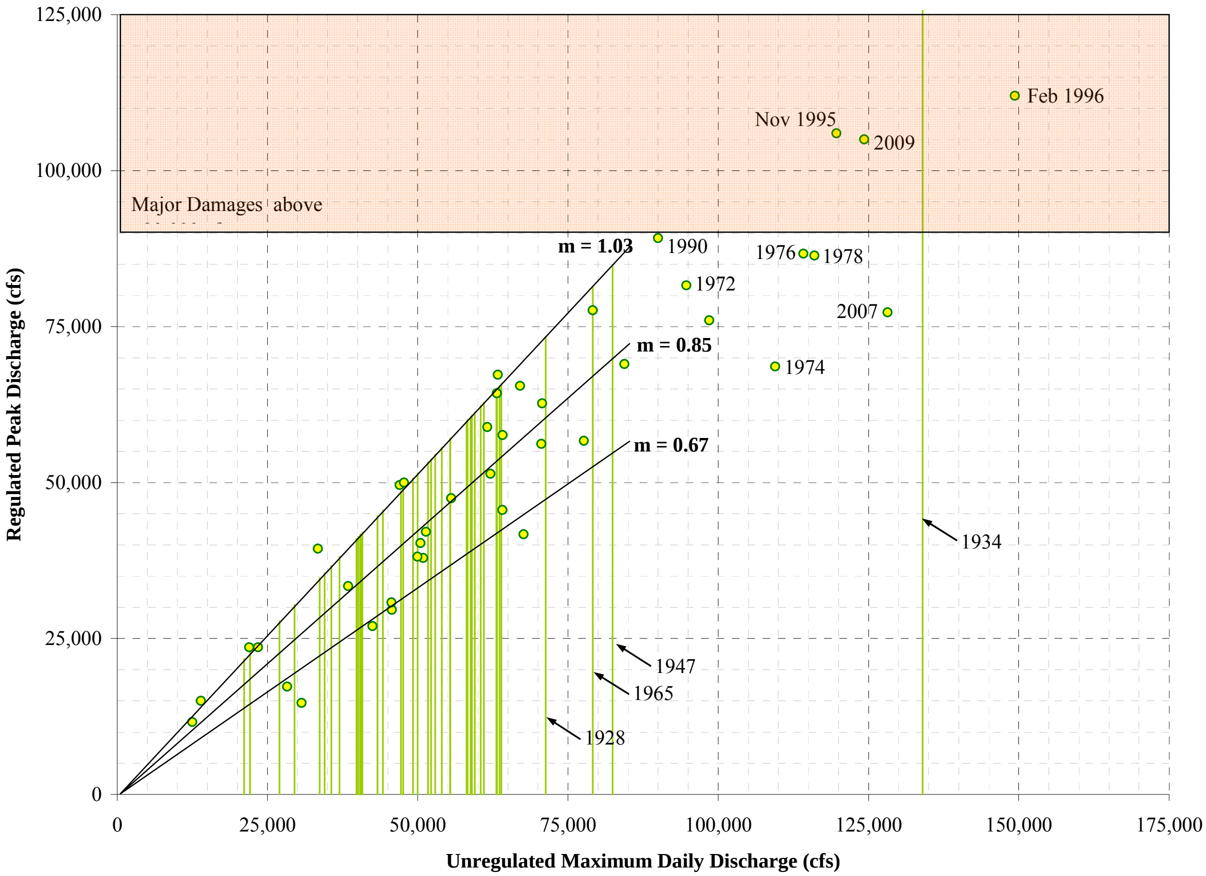


Figure B. 9. Regulated-unregulated relationships used to estimate regulated peak discharges for pre-regulation years.

Figure B. 10 shows the expanded regulated discharge data as described above, ranked and plotted on a log-normal scale. The figure includes three different series from the three regulated-unregulated relationships. Plotting positions were recalculated for each series. The figure also shows unregulated peak discharge-frequency curve (described later) and the discharge-frequency relationship of only the measured data from 1969 to 2009. The December 1933 event is plotted assuming a regulated peak of 110,000 cfs, but an additional series was created to depict the range of 100,000 to 120,000 cfs.

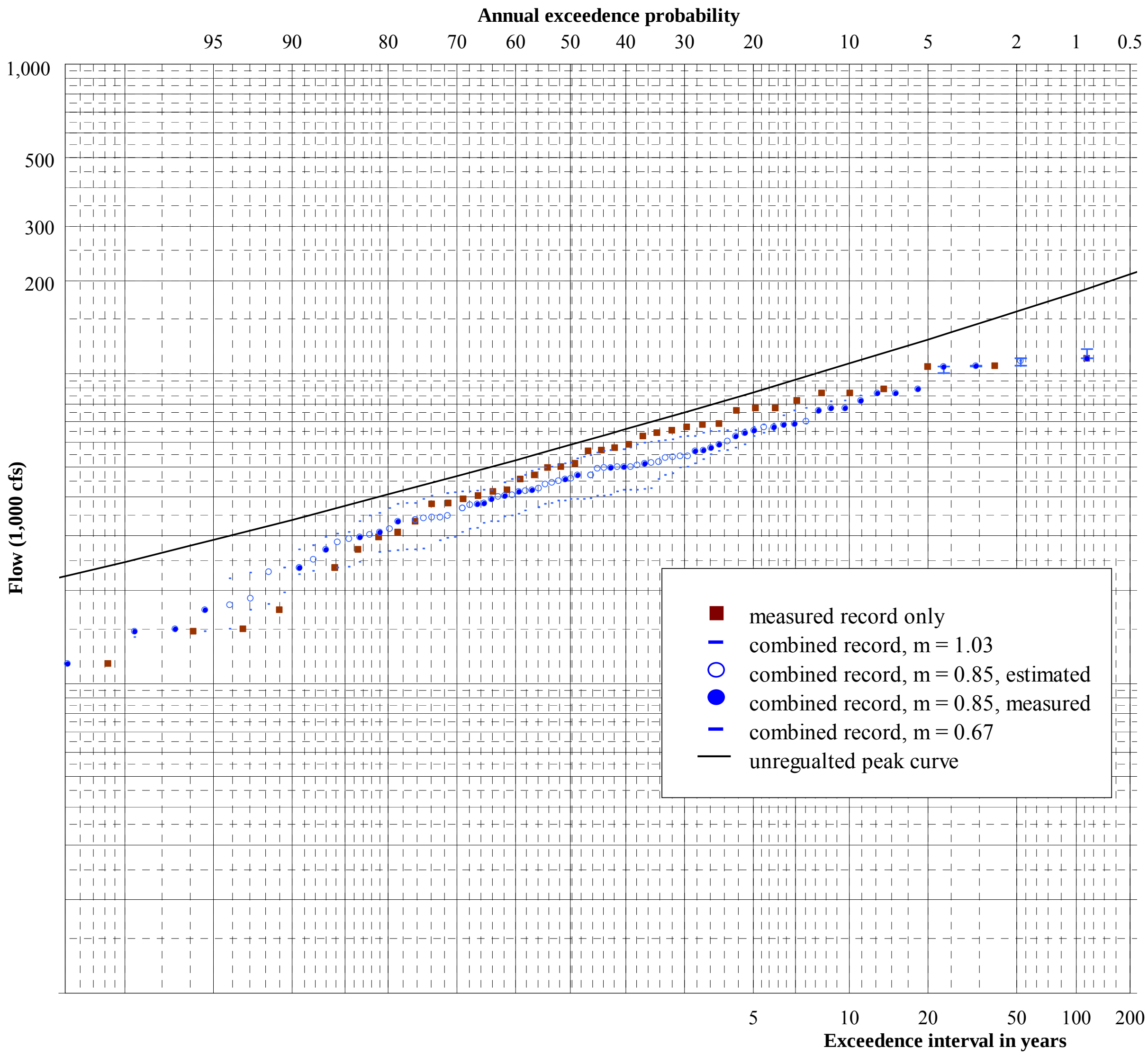


Figure B. 10 Regulated frequency curves using various regulated-unregulated discharge relationships to estimate flows less than 90,000 cfs

While there is considerable uncertainty with this method of estimating regulated flow, it is assumed that the range of possible regulated frequency curves is within the maximum (effectively no regulation) and minimum (optimum regulation) curves, and that middle curve, calculated using the best-fit regulated-unregulated relationship, is an adequate approximation of the median regulated discharge-frequency relationship for the 83 year POR. How these curves affect the final regulated frequency curve is discussed later in the report.

B.3.4. Historic Data

Historic data are used to define historic periods that affect the plotting position of measured results and the discharge-frequency statistics. Incorporating a well-researched historic period into the analysis also reduces the uncertainty of the estimated discharge-frequency analysis.

The 1997 Cowlitz River Flood Hazard Study discussed historical flood data from "an old Corps of Engineers frequency curve" and from the Centennial edition of the Longview Daily News in 1953. Noteworthy events that occurred prior to the Castle Rock gage include floods in 1896, 1906, 1917, and 1923. The flood of 1896 was described at the time as the greatest flood of recorded history. The peak discharges of the 1906, 1917, and 1923 events were estimated at between 45% and 60% of the February 1996 flood peak, and all three were less than the 139,000 peak discharge of the December 1933 flood. It was concluded in the 1997 report that it would have been very unlikely that there were other floods between 1896 and 1927 greater than the unregulated February 1996 event, and therefore a historical period of 1896 to 1996 was used in calculating the unregulated peak frequency curve. As it was not relevant to the methods used at the time, the 1997 report did not discuss assumptions about volume-frequency or regulated discharges.

To assess historical period assumptions to apply for the volume frequency curves (discussed in the following section), it is helpful to look at the largest volume-frequency flows. The largest 1- through 4-day average discharges occurred with the February 1996 event, and all of these are greater than the peak discharge of the 1906 flood, the largest known flow in the period between 1896 and 1927, estimated at 112,000 cfs. The largest 5-, 7-, and 15-day discharges (106,000, 96,000 and 75,000 cfs, respectively) occurred with the December 1933 event. Looking at the large, long-duration events that have occurred in the gage record, the 1947 event stands out as being an unusually long-duration event with a relatively small peak discharge. With this event, the ratios of 5- and 7-day average discharges to peak discharge are 0.88 and 0.77, respectively. If the 1906 event had a similar shape, the 5- and 7- day durations would have equaled 98,600 and 86,200, both of which are lower than the corresponding discharges that occurred with the December 1933 event. The average discharge of 75,000 with the December 1933 flood is a significant high outlier, and 40% greater than the next highest 15-day average discharge occurring with the December 1978 flood event. It also should be noted that both of these events (December 1933 and December 1978) were comprised of two, large, back-to-back flood waves, as opposed to a sustained moderately high discharge.

The peak unregulated discharge for the 1906 flood, the largest known flow in the period between 1896 and 1927, was estimate to be 112,000 cfs, which is approximately the same discharge as the regulated peak of the 1996 event. Assuming at least some flood reduction would have occurred with this event, the regulated peak would be less than that of the February 1996 event, and is fair to assume the same historical period applies for the regulated peaks as with the unregulated.

B.4. SYNTHETIC HYDROLOGY ANALYSIS

Synthetic hydrology is used to estimate the upper end of the regulated discharge-frequency curve beyond the systematic record. Several spatial and temporal storm patterns based on historic events are used to create not one but a suite of hypothetical storms for each of the following design annual exceedence probabilities (AEP): 0.10, 0.02, 0.01, 0.005, and 0.001. The process of creating synthetic floods involves 1) creating unregulated peak and volume-duration frequency (VDF) curves at Castle Rock and at the main tributaries, 2) identify patterns in storm intensity from historic events, 3) translate storm patterns to inflow

hydrographs, and 4) simulate reservoir regulation using HEC-ResSim. With a storm pattern analysis, it is recognized that a given n-event regulated peak discharge at Castle Rock is not attributed to one specific storm configuration but is a composite of many different possible storms. The approach taken in this study is to run a multitude of storm configurations for a given n-event to provide a better resolution on hydrologic possibility and some understanding of variability and certainty.

B.4.1. Unregulated Discharge-Frequency Curves

Peak discharge and VDF curves are used to link measured discharge to AEP, which for the purposes of this study is considered to be a normalized measure of storm intensity, and then from design AEPs back to discharges. For this analysis, unregulated peak and VDF curves are created for the following locations:

A. Castle Rock
B. Upper Cowlitz Basin
C. Tilton Basin
D. Toutle Basin
E. Castle Rock Locals
F. Reservoir Locals

The VDF curves at Castle Rock are used to define flood volumes for a given design AEP flood. Any hypothetical storm event should produce the design volume at Castle Rock after all of the hydrographs from the tributaries are routed to the downstream control point. The tributary VDF curves serve two purposes: they are used 1) to help identify spatial variability in storm intensity across the entire basin, and 2) to re-distributing flood volumes from Cowlitz River at Castle Rock throughout the basin.

Log Pearson type III distributions were fit to the peak, 1-, 2-, 3-, 4-, 5-, 7-, and 15-day annual maximum discharges per guidelines specified in Bulletin 17B, including high/low outlier tests, historic period adjustments, and conditional probability adjustments. Low outlier thresholds were applied to create a better fit in the upper end of the curves. The final adopted statistics for a given location were chosen to create smooth transitions between the different VDF curves. In no cases were regional skew values applied to calculate weighted skews for peak flood frequency curves; it was noted that the skew values for each basin were reasonable and consistent with trends in skew values with the other VDF curves, as well as with the regional skew values of 0.3 and 0.0 for the two polygons covering the Cowlitz basin. The data, historic data assumptions, flood frequency statistics, and curves are attached at the end of the report. Annual maxima attributed to predominantly snowmelt-driven floods were screened for use in this analysis, for the purpose of maintaining homogeneity.

B.4.1.1. Castle Rock

The peak and VDF curves for Cowlitz River at Castle Rock were created using a combination of measured data from the pre-regulation period (1927-1968) and modeled data for years where flows were significantly impacted by Mossyrock Dam (1969-2009). Measured data included annual maximum instantaneous discharges and daily data from the USGS website. Duration maxima were calculated manually using Excel.

For years following the completion of Mossyrock Dam, detailed simulations were performed to estimate without-project or unregulated discharges. CENWP regularly computes unregulated discharges as a part

of annual monitoring. The HEC-ResSim model described earlier is used for these simulations. Inflow hydrographs at Cowlitz River near Randle (Upper Cowlitz), Tilton River, and the Toutle River are created first using available short-duration data, and then daily data are used in the absence of hourly data. Locals were calculated directly for most years. Changes in reservoir storage are calculated using twice-daily data.

B.4.1.2. Upper Cowlitz Basin

The VDF curves for Upper Cowlitz are created primarily using annual peak and daily flow data from the Cowlitz River near Randle, WA gage (#14233400) from the USGS website. Coinciding with the construction of the Cowlitz Falls dam, the Cowlitz River near Randle gage was abandoned in 1994; however, the Cowlitz River near Kosmos, WA (#14233500) was in place just downstream. The drainage areas are within 1% of eachother, so no adjustments were made to the Kosmos gage data. The Cowlitz River at Mossyrock, WA gage (#14235000) is used to add an additional nine years between 1927 and 1948 when the gage was installed the Cowlitz River near Randle site. This was done using correlations of duration maxima for a period of overlapping data from 1948 to 1960.

B.4.1.3. Tilton Basin

The VDF curves for the Tilton Basin are created using peak and daily discharge data from the existing gage at the Tilton River above Bear Canyon near Cinebar, WA (#14236200) from WY 1957 to the present, and from the old Tilton River near Cinebar, WA (#14226500), in place from WY 1942 to 1957, available on the USGS website. A drainage area ratio (141/156) was applied to the latter to create a continuous dataset data set of 68 years.

B.4.1.4. Toutle Basin

The data used for the Toutle Basin VDF curves includes the entire period-of-record of both the Toutle River at Tower Road, WA gage (#14242580) and the Toutle River near Silver Lake, WA gage (#14242500) minus the five years following the Mount St. Helens eruption (WYs 1981 – 1985) for a total of 81 to 79 years. For the homogeneous dataset, the Tower Road gage data is adjusted to the Toutle River near Silver Lake gage using the drainage area ratio (474/496).

To determine if the sediment retention structure (SRS) on the North Fork Toutle has an impact on the peak at Tower Road, timing of available peak flows were compared. The USGS gage #14240525, NF Toutle River below SRS near Kid Valley WA, was in service for WYs 1990-1998 and 2001-2002. Additionally, hourly data for the Tower Road gage is available for most of this period. Comparisons of peak flow at the Kid Valley gage and the Tower Road gage for the 5 years of matching data showed that in all but one case, the peak at the Tower Road location occurred on the day prior to the peak at Kid Valley. It was concluded that since the North Fork Toutle is peaking after Tower Road, the North Fork (and therefore, the SRS) does not impact the peak at Tower Road, and a detailed study of the SRS was not warranted. However, it should be noted that the actual time of the Kid Valley peak was not available. In 1996, Kid Valley and Tower Road peaked on the same day. The peak at Tower Road was midday. Since the hour of the Kid Valley peak was not available, it was not possible to determine the timing relationship. However, since the Tower Road peak occurred at mid-day, there is a 50% chance that the Kid Valley peak occurred after the Tower Road peak.

B.4.1.5. Castle Rock Locals

Castle Rock Locals VDF curves were created using calculated data from a simple water balance. Daily data were calculated by subtracting the measured average daily discharges at Castle Rock from the two

upstream gages at Cowlitz River below Mayfield and Toutle River near Silver Lake. With travel times of 2 and 7 hours from Tower Road and Mayfield to Castle Rock, respectively, no routing or lag was applied to the daily data. Recognizing that the daily data calculated using such a course water balance is inadequate for estimating maximum annual daily discharges, it is assumed that over-time, the duration maxima could be useful, perhaps at 3- or 4-day and larger. The daily data were compared with detailed hourly estimates of locals performed in the 1997 study. It was confirmed that the annual maxima from the daily series was not usable, and the 2-day maxima were indeed questionable; however, the 3-, 4-, and 5-day maxima were showed some decent correlation, averaging 9%, 5% and 3% less than the hourly estimates, respectively. The daily data were adjusted by these factors and then used to estimate flood frequency statistics.

Frequency analyses were then performed to create Log Pearson III VDF curves for the 3- through 15-day durations. The statistics from the VDF curves were compared with the VDF curves statistics from the other tributaries. The mean, standard deviation, and skew values for the peak, 1-, and 2-day curves were adjusted assuming a smooth continuation of the statistics with increasing/decreasing duration, similar to those from other tributaries.

As a check, the hourly hydrographs for the Castle Rock Locals calculated in the 1997 study were reduced to duration maxima and translated to AEPs using the VDF curves described above. These AEPs were checked against the AEPs of the other tributaries (discussed in more detail in the following storm patterns analysis section) to ensure that it was similar in magnitude, not consistently high or low. The flow frequency statistics were then adjusted slightly to obtain more realistic AEP curves in the various historical events.

(Note - It is agreed that the daily data from which these annual maxima and VDF curves from are not ideal, but it is believed that these data provide a better estimate of the hydrologic characteristics of sub-basin than those created using the drainage area method applied to the Toutle River basin. The drainage area method was not used in the present study due to several factors, including the large drainage-area ratio required to extend the record of gaged area to the total area data (838/474 = 1.77), the large difference in average annual precipitation (84 inches for the Toutle Basin and 53 inches for the Castle Rock Locals), and the large difference in basin characteristics, such as slope and location. The drainage area method would produce the exact same VDF curves for the two sub-basins, only the mean would be shifted by a factor equivalent to the log of the drainage area ratio. By using actual, albeit course data, differences are evident in the standard deviation and skews of the VDF curves, indicating substantially different basin characteristics beyond size, as would be expected.)

B.4.1.6. Reservoir Locals

The VDF curves for Reservoir Locals were estimated using the adopted statistics from the other VDF curves in this study. Due to the size, location, topography, and average annual precipitation (AAP), the basin was assumed to behave somewhere between that of the Tilton basin and the Castle Rock; therefore, the standard deviation and skew values for each duration were calculated as the average of Tilton and Castle Rock Locals standard deviation and skews. The mean values were calculated using empirical relationships relating mean to drainage area and AAP. The following relationship worked well in producing strong correlations:

$$\text{Mean } \log Q = \log(DA * AAP^{x}) \qquad \textit{Equation 3}$$

Where mean log Q is the mean of log discharges, DA is the drainage area is square miles, AAP is the average annual precipitation in inches, and x is an AAP weighting factor. The log of the product of DA

and AAP^x was calculated for each sub-basin, and x was varied to create the best-fit polynomial regression through the sub-basins. The final relationships are shown graphically in Figure B. 11. The x-values used are shown in Table B. 2.

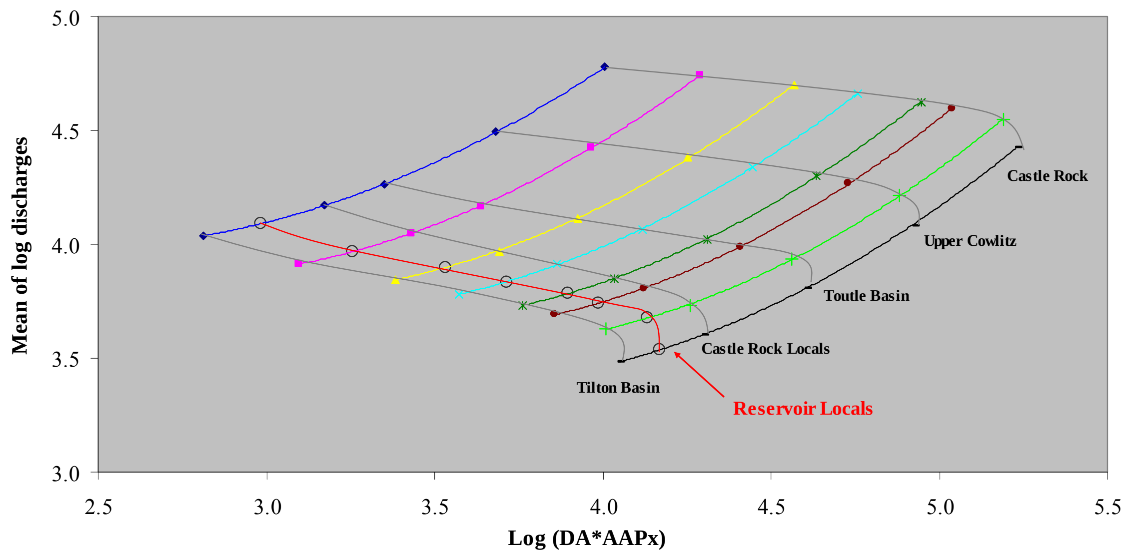


Figure B. 11. Relationships used to estimate the log mean discharge at Castle Rock Locals and Reservoir Locals from drainage area and average annual precipitation.

Table B. 2. X-values used to calculate mean of log discharges for Reservoir Locals.

Duration	X-value
peak	0.35
1	0.50
2	0.65
3	0.75
4	0.85
5	0.90
7	0.98
15	1.00

B.4.2. Storm Patterns Analysis

Seventeen historical rainflood events were analyzed for spatial and temporal variability in storm intensity. Peak and duration maxima were collected for all 17 historic events and AEPs were estimated using the peak and VDF curves described in the previous section. Plots were created for each historic storm event showing the AEP for each duration at each tributary. Also included in the plots are the frequency of the regulated peak discharge, approximated using the Weibull plotting position. Figure B. 12 is an example of one of these historical event plots. All 17 historic event plots are attached at the end of the report.

Note: For Castle Rock, maxima were calculated using hourly data wherever possible, instead of day-end averages, and AEPs were calculated using adjusted flood frequency statistics that account for the increase in discharges resulting from this calculation method. This was done because day-end averages would not suffice when balancing the routed hydrographs discussed in the next section.

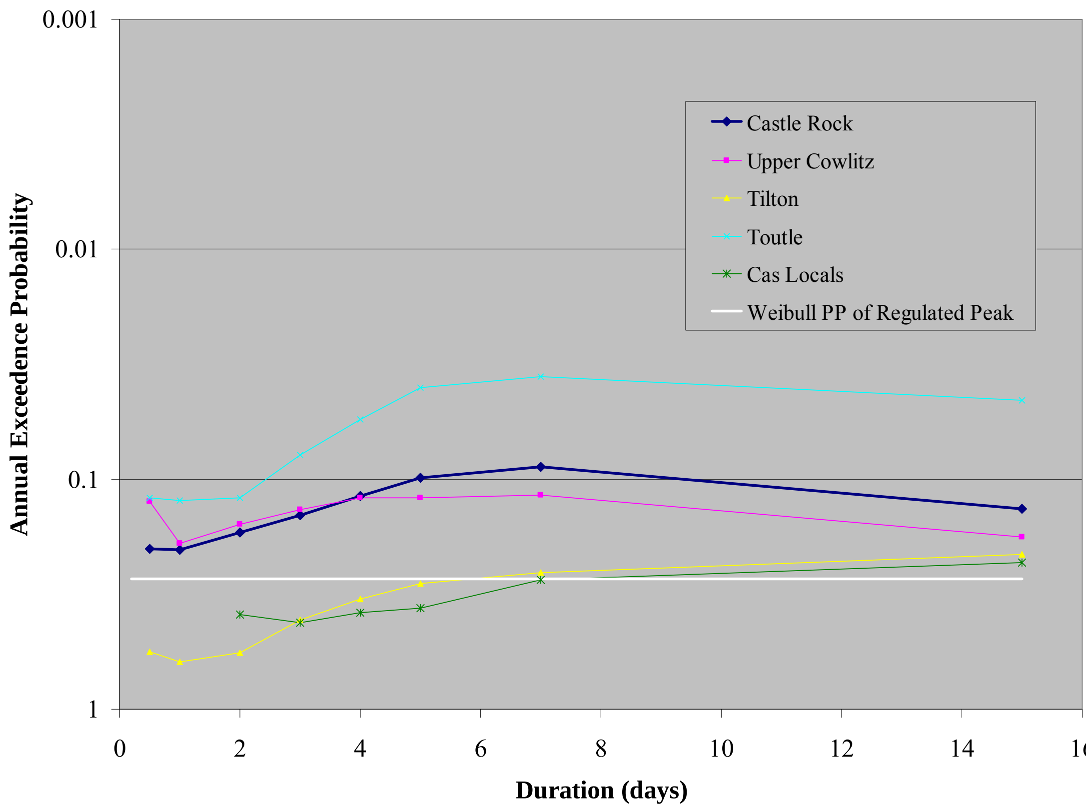


Figure B. 12. Example of historic event plots used to identify temporal and spatial variability in storm intensity. This particular event is the WY 1997 event, a longer duration event concentrated in the higher elevation tributaries, more specifically in the Toutle Basin.

One thing apparent when looking at all of the historic event plots is the wide range of spatial and temporal patterns as well as how the AEPs of the tributaries and unregulated discharge at Castle Rock relate to the regulated peak AEP. With the end-goal of creating a set of hypothetical storms for a design AEP with which we can estimate a range of peak regulated discharges at Castle Rock, it is necessary to create a process for defining a given event with a single representative AEP. In the 2002 Sacramento and San Joaquin Comprehensive Study, the 3-, 5-, and 7-day AEPs were averaged to create a simplified representation for a given storm event, with each main stem location or tributary having exactly one AEP. In the much smaller Cowlitz Basin, the average of the 1- through 3-day AEPs is a better approximation of the intensity of the event as relevant to regulated peak discharges. This assumption applied to the 1997 event, shown in the previous figure, results in substantial difference with the average AEP of the 1-, 2-, and 3-day durations equal to 0.17 and the approximate regulated peak equal to 0.26. This exercise applied with each of the historic events illustrates the inherent difficulty in creating and defining synthetic floods based on unregulated flood frequency and the flawed but necessary assumption that the regulated flood frequencies are directly related to unregulated flood frequency.

To begin to quantify temporal variability, the average of the 1-, 2-, and 3-day AEPs were calculated for each of the historic events for all locations. The relative AEP for the peak and individual durations were calculated by dividing by the average AEP, and then all of the storm events were plotted on one graph for each tributary (excluding Reservoir Locals). From this array of curves, three patterns were created to represent the possible temporal shapes, all of which have an equal possibility of occurring. The three

patterns represent short, medium, and long duration storms. Figure B. 13 shows the patterns for the Cowlitz River at Castle Rock.

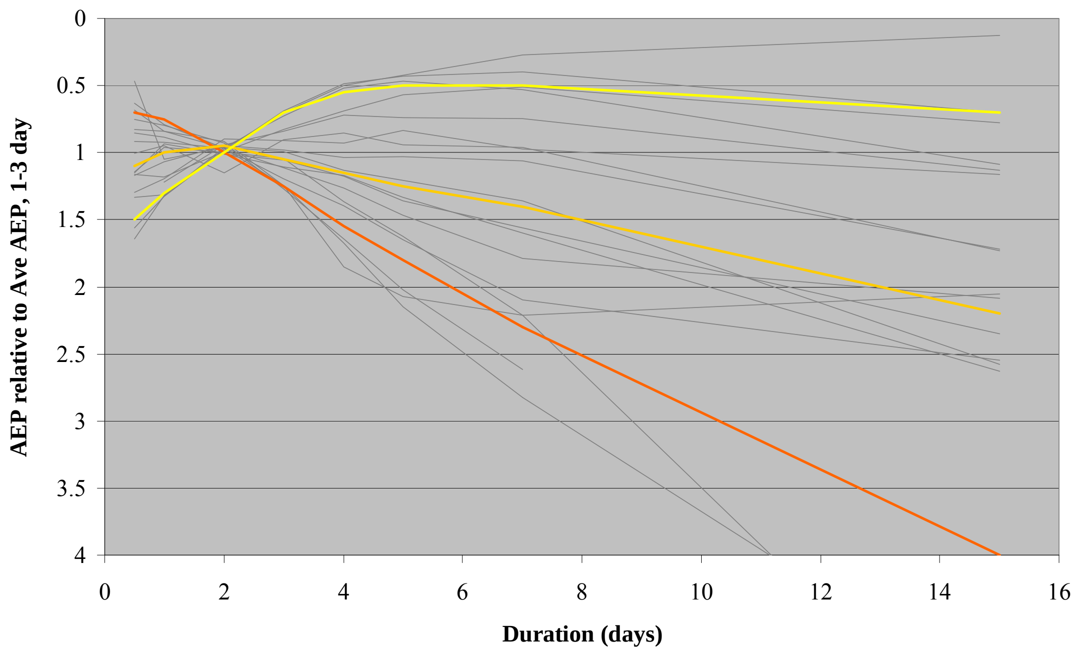


Figure B. 13. The three patterns used to describe different duration patterns at Castle Rock. The red, orange, and yellow lines represent the short, medium and long duration patterns, respectively. Individual historic flood events are in gray behind the five patterns. Note, a synthetic analysis disregarding temporal variability would essentially assume a flat pattern at relative AEP equal to 1.

To identify patterns in spatial variability, the average AEP at each tributary is divided by the average AEP at Castle Rock. This term "relative AEP" is useful in identifying the intensity in one tributary relative to another. It also creates an easy metric for distributing a storm pattern based on design AEP at Castle Rock. Figure B. 14 shows relative AEPs of the tributaries (excluding Reservoir Locals) for the 17 historic events. Storms that are fairly uniform in intensity will be clustered near 1.0, and conversely, a storm that is largely concentrated in one or two tributaries will have a wide spread in relative AEPs. The figure also shows the Castle Rock AEP and the regulated peak discharge frequency estimated using the Weibull plotting position.

One thing evident in Figure B. 14 is, again, a high degree of variability in spatial patterns of storms. The 1978, 1976 and November 1995 storm events could be described as being clearly concentrated in the Upper Cowlitz Basin, while the February 1996 and 1947 events clearly represent storms centered in the Toutle Basin. Another type of pattern could be described as highland centered, with low relative AEPs in Upper Cowlitz and Toutle (2003 and 1997), versus lowland centered (2009, 1990, and 1972), which all produced remarkably large regulated peak discharges at Castle Rock when the average Castle Rock AEPs were fairly small. There does not seem to be a strong correlation between Castle Rock AEP, duration, and centered location, with the exception that all of the lowland centered storms had short durations.

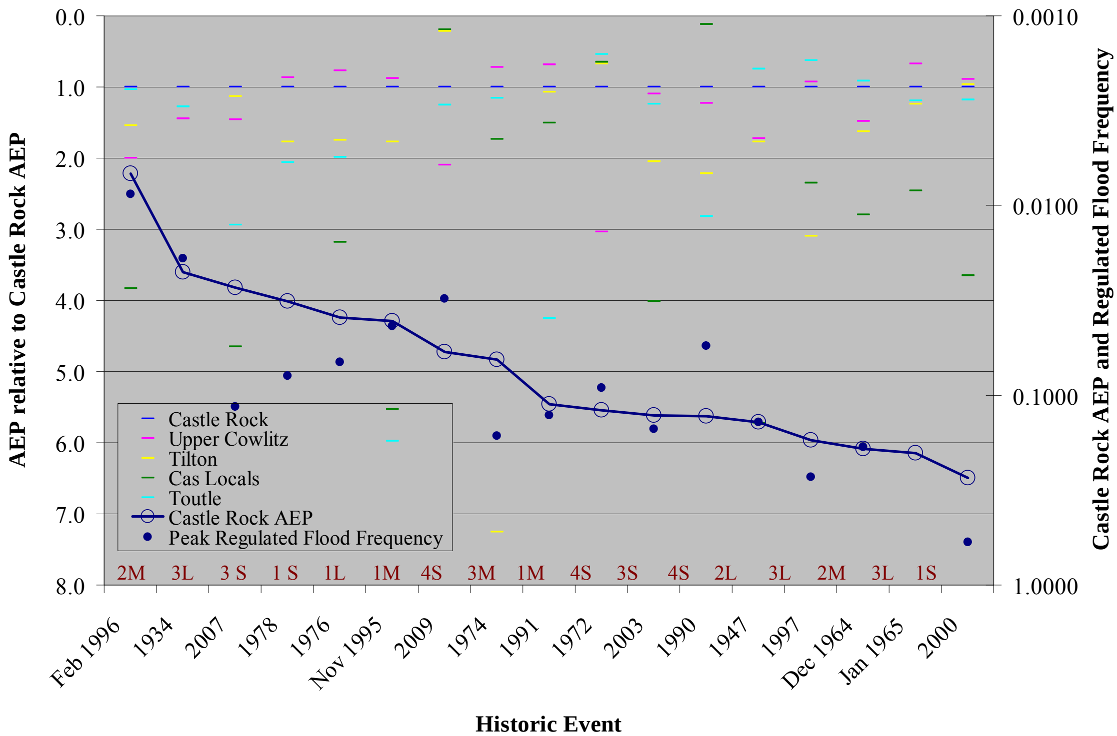


Figure B. 14. Spatial variability in storm intensity, shown here as the tributary AEP relative to the AEP at Castle Rock. The events are sorted from largest to smallest, left to right. Letters at the bottom indicate the duration pattern of the event (S, M, and L signify short, medium, and long duration temporal patterns), and the number indicates the associated spatial pattern, described in the following paragraph.

To describe the range of spatial patterns seen historically, 4 patterns were created that relate tributary AEP to the Castle Rock AEP, shown in the Figure B. 15. A design AEP would be defined at Castle Rock and these patterns would then be used to define the average AEP at each tributary. The AEPs for a given duration are then calculated using a corresponding duration pattern as described earlier.

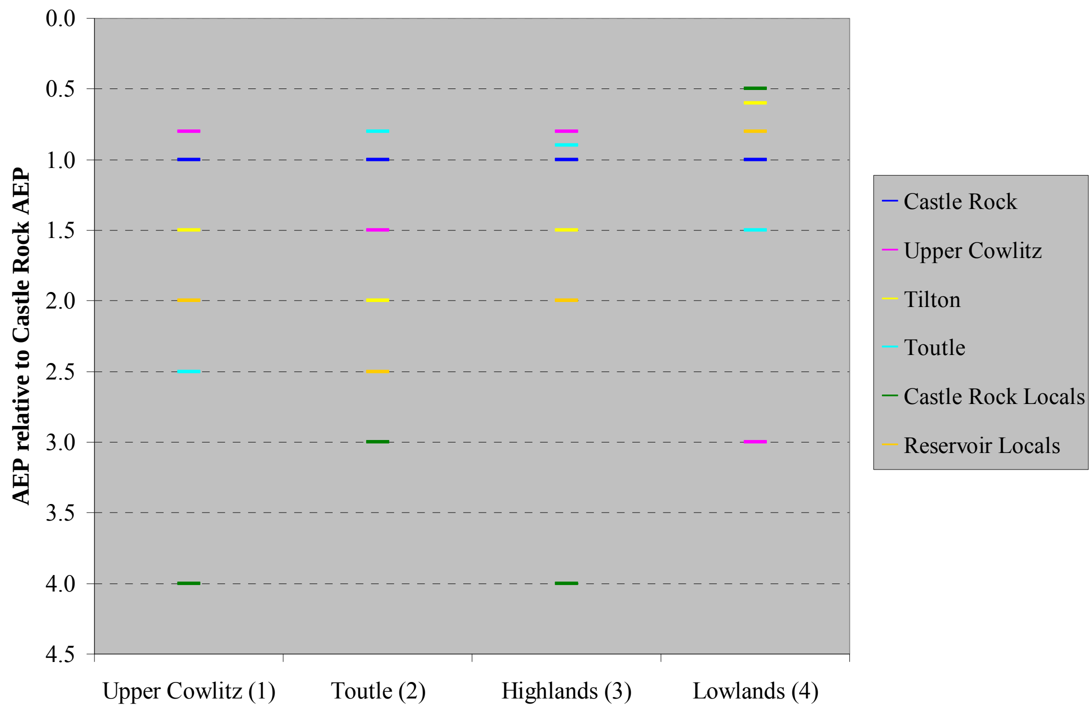


Figure B. 15. The four spatial patterns used to describe possible spatial variations in storm intensity.

The final configuration of AEPs, or a "centering", would typically be different than the initial configuration created from the design AEP, spatial pattern, and temporal pattern due to the need to create balanced hydrographs that produce the desired, design incremental volumes at Castle Rock after being routed. The hydrographs and how they are balanced is described in the following section. Figure B. 16 is a graphical example of a final, balanced centering. All of the final centerings are attached at the end of the report.

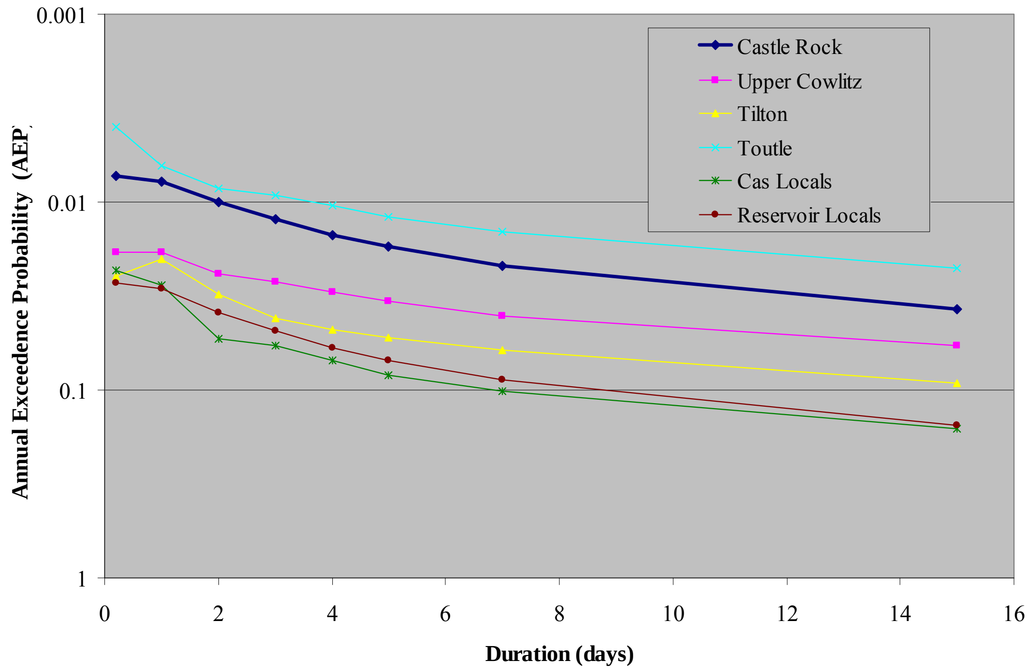


Figure B. 16. Graphical centering of a 0.01 AEP, Toutle-centered, short duration event.

B.4.3. Synthetic Hydrographs

Translating a centering to individual tributary hydrographs is done similarly to the methods applied in Hickeyet al's Sacramento San Joaquin Comprehensive Study, only scaled down for the smaller basin. A single 7-day wave is used for the hypothetical floods in this study. This duration is based on the shape and timing of large historic rain floods in the basin, which typically were seen at Castle Rock in the form of 5- to 7-day flood waves.

Flood volumes are translated to the synthetic hydrographs using incremental volumes from the VF curves. The 1-, 2-, 3-, 4-, 5-, and 6+7-day discharges are converted to volumes by multiplying the average discharge by the duration, and then converting to incremental volumes by subtracting the 1-day from the 2-day, the 2-day from the 3-day, etc. These incremental volumes are assigned to different 12-hour periods in a generic hydrograph. The peak discharge was applied directly in the middle of the largest 1-day flow period. The periods were smoothed to create a more natural hydrograph, but the incremental volumes were still adjusted to be equal to or greater than the design volumes by a maximum 4% (based on the difference between 24-hour versus day-end maxima, as discussed earlier in the Castle Rock unregulated frequency curve section.)

The timing of the peak for each tributary is based on one of three temporal patterns, each one representative of short, medium, and long duration events from the historic record. The three patterns, shown in Table B. 1, are defined timing of the peak relative to the peak on the Tilton Basin, which was consistently the first gage to peak.

Table B. 3. Relative time-of-peak in hours for the different duration patterns.

	Upper Cowlitz	Tilton	Toutle	Castle Rock Locals	Reservoir Locals
Short	8	0	1	1	0
medium	16	0	6	2	0
Long	24	0	12	6	0

All tributary hydrographs are routed to Castle Rock using a coarse water balance approach in Excel, using basic smoothing and lag. The maximum 24-hour discharge at Castle Rock is compared to the design AEP 24-hour discharge. The spatial pattern is adjusted slightly until the routed volume is within 2% for the 1- through 5-day durations. The duration patterns are then adjusted slightly to ensure a smooth hydrographs (i.e. the 5-day incremental volume cannot be greater than the 4-day incremental volume). The spatial pattern is then adjusted again as necessary in an iterative process to achieve a smooth and balanced set of hydrographs. Figure B. 17 is an example of a balanced synthetic hydrograph.

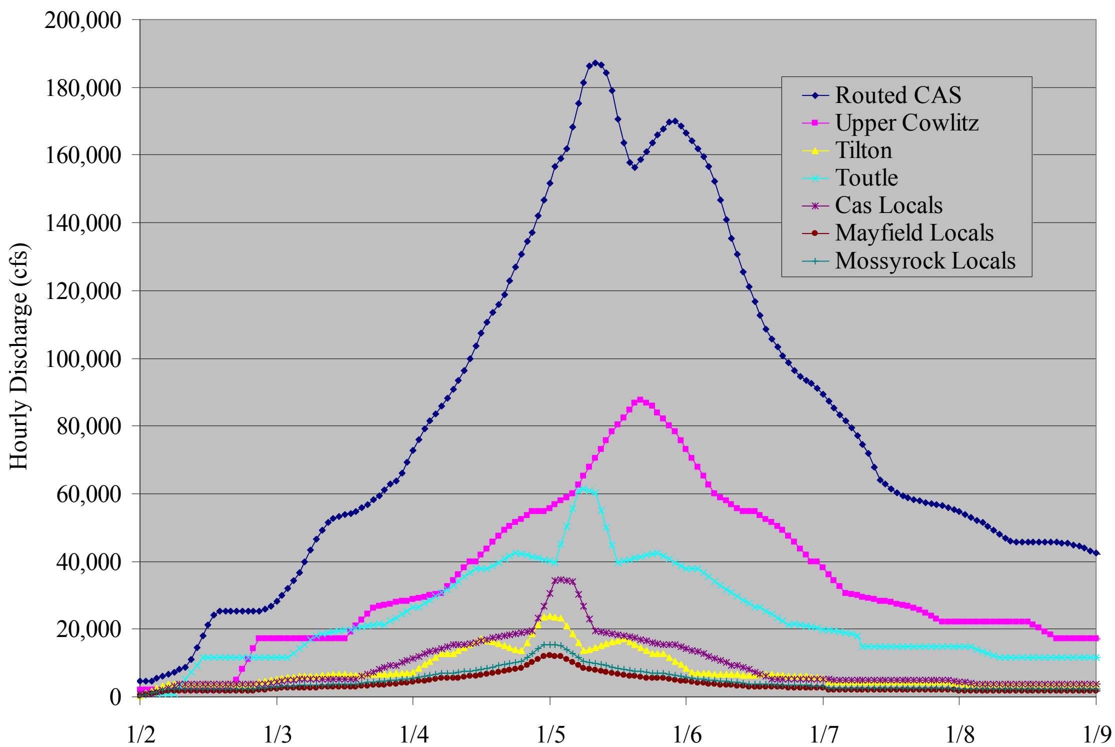


Figure B. 17. Tributary hydrographs and a coarsely-routed Castle Rock hydrograph of a medium duration, Toutle-centered, 0.01 AEP event.

B.4.4. Reservoir Routing

The calibrated HEC-ResSim model is used to route the synthetic hydrographs. Routing coefficients are calibrated to match observed attenuation and lag using hourly data from multiple recent storm events. Reservoir regulation is simulated for Mossyrock Dam only; the 21,000 acre-feet of storage capacity behind Mayfield Dam was assumed negligible compared to the 360,000 acre-feet behind Mossyrock. The key factors affecting reservoir operations at Mossyrock Dam are the downstream flow at Castle Rock and the amount of storage available in Riffe Lake. Per guidelines in the water control manual, the reservoir is to operate to keep Castle Rock at or below 70,000 cfs. Additional criteria are added to simulate special curve operations when approaching the top of flood control space.

A starting elevation equivalent to the design winter flood control pool (745.5') was chosen for each synthetic storm. This is a conservative estimate considering that the lake is typically well below the rule curve during the extended rainflood season (November through February), averaging about 10 feet below the winter flood control pool (December through January). It is noteworthy that the vast majority of annual maximum discharges occur between November and February, and all of the large rainfloods have occurred in this time period. Figure B. 18 shows the lake elevation non-exceedence statistics for years WYs 1975-2009.

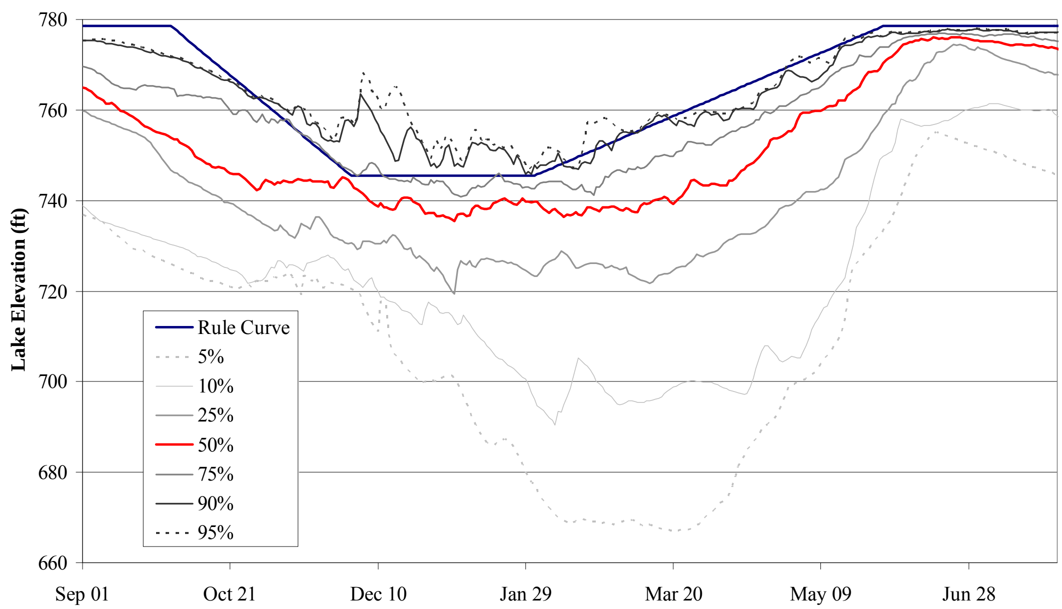


Figure B. 18. Riffe Lake elevation statistics show that the reservoir is typically below the rule curve for the entire flood season.

To reduce the amount of model runs, each set of tributary hydrographs for each spatial and temporal pattern for a given n-event were combined into one continuous hydrograph of multiple flood waves, each wave representing a different storm pattern. Ample time was provided between flood waves to allow for the reservoir to evacuate any stored water. Also included for each n-event flood wave series was a uniform flood event (no temporal or spatial variability).

B.4.5. Synthetic Hydrology Results

Both the regulated and unregulated hydrographs at Castle Rock calculated for each simulation (one per n-event) were output from the model. The peak and duration maxima were calculated for each unregulated pattern/flood wave and converted to AEP. These AEPs were always slightly different than the design AEP, due to the inability to balance the hydrographs exactly in Excel. These re-computed AEPs were used in plotting the results.

The regulated hydrographs for each flood wave were interpreted individually, rather than simply choosing the maximum value for each flood wave. Peak discharges were consistently related to either the locals downstream of the reservoir or a later peak caused by the reservoir reaching capacity, at which time the reservoir outflow equals the inflow. Where the maximum discharge resulted from the locals, the peak value was assumed to equal the regulated discharge for that event. When the highest peak occurred with the reservoir-filling peak, some discretion was used to estimate a realistic regulated peak discharge for the event. In cases where a large decrease in discharge would separate a large local peak and an even larger reservoir-filled peak, more realistic regulated hydrographs were assumed to estimate a minimum and occasionally a maximum regulated peak discharge. Figure B. 19 is an example of choosing both minimum and maximum regulated peak discharges from the model-output hydrographs.

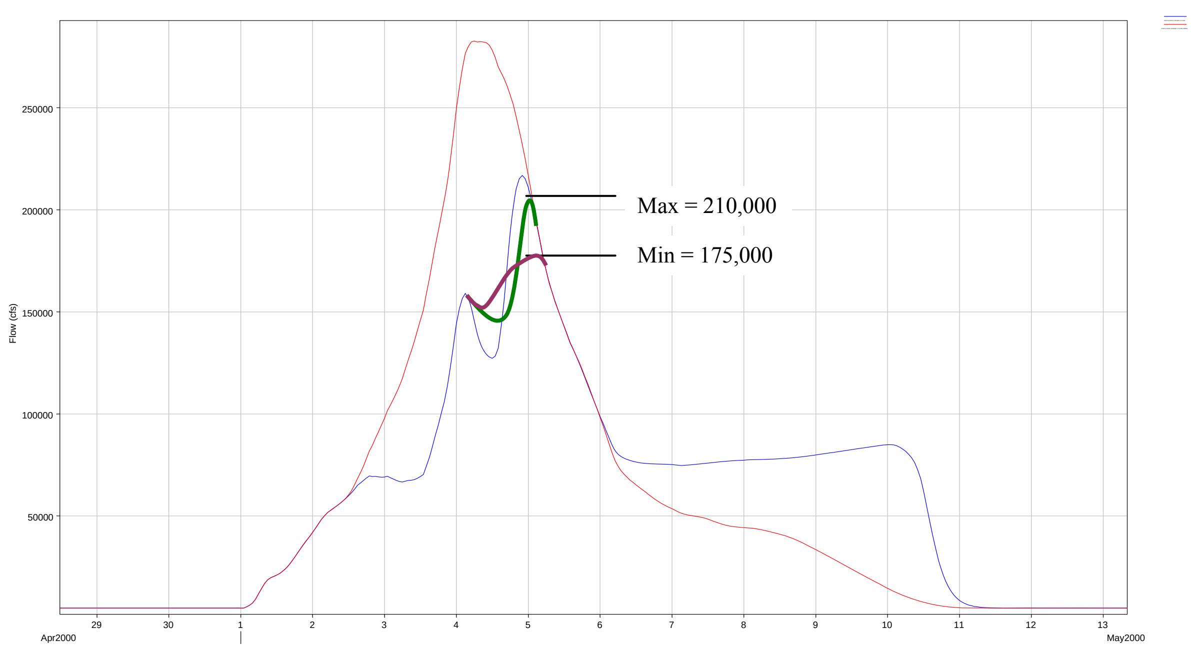


Figure B. 19. Regulated (blue) and unregulated (red) hydrographs for the 0.001 AEP, "Uplands" centered, short duration pattern and the range of realistic regulated peaks. In this case, an average value of 193,000 would be the chosen regulated peak discharge for specific event.

For the 0.10 and 0.02 AEP events, the maximum discharge for a given flood wave was often a result of a third type of peak attributed to simulated, unrealistically aggressive drawdown following the local peak. These peaks were discounted because it is assumed that reservoir storage would not be evacuated at a rate higher than the peak discharge resulting from the downstream locals. In these cases, the maximum discharge would be taken from the local peak.

The final regulated peak discharges for all of the synthetic storms are shown in Table B. 4. The plotting position of the individual hypothetical storms, shown in italics in Table B-III, is equal to the minimum of the 1- through 3-day AEPs at Castle Rock, assuming that the defining intensity for a given hypothetical event coincides with the defining temporal pattern, i.e. the 1-day intensity is relevant to the short duration pattern, and the 3-day intensity is relevant to the long duration pattern.

Table B. 4. Simulated regulated peak discharges (cfs) for all of the hypothetical storms.

Design	Upper Cowlitz			Toutle			Uplands			Lowlands	Uniform
AEP	short	medium	long	short	medium	long	short	medium	long	short	none
0.1	68,441	67,744	70,064	77,934	71,014	72,836	70,072	69,176	71,868	94,606	68,333
	0.0819	*0.0980*	*0.0815*	*0.0836*	*0.0974*	*0.0791*	*0.0883*	*0.0965*	*0.0793*	*0.0760*	*0.0975*
0.02	88,084	72,494	73,032	111,141	94,833	86,571	92,377	79,494	74,401	125,561	93,678
	0.01577	*0.01868*	*0.01455*	*0.01568*	*0.01864*	*0.01480*	*0.01613*	*0.01900*	*0.01425*	*0.01472*	*0.01887*
0.01	98,868	85,510	111,364	125,890	110,622	100,916	109,770	91,459	105,682	143,521	107,055
	0.00771	*0.00906*	*0.00664*	*0.00762*	*0.00910*	*0.00726*	*0.00778*	*0.00931*	*0.00695*	*0.00703*	*0.00937*
0.005	109,764	109,772	141,406	141,817	121,948	120,557	121,134	107,214	143,552	160,343	122,427
	0.00381	*0.00426*	*0.00331*	*0.00379*	*0.00443*	*0.00343*	*0.00383*	*0.00439*	*0.00327*	*0.00349*	*0.00437*
0.002	163,060	170,496	198,264	164,488	145,790	168,813	157,145	168,484	196,192	184,258	157,511
	0.00147	*0.00169*	*0.00123*	*0.00145*	*0.00168*	*0.00129*	*0.00147*	*0.00169*	*0.00121*	*0.00137*	*0.00170*
0.001	190,000	217,622	236,114	180,430	185,000	206,755	192,500	205,000	231,080	203,861	187,500
	0.00071	*0.00080*	*0.00059*	*0.00071*	*0.00081*	*0.00060*	*0.00072*	*0.00084*	*0.00060*	*0.00066*	*0.00083*
pattern	1-1	1-2	1-3	2-1	2-2	2-3	3-1	3-2	3-3	4-1	5
weight	12%	12%	6%	0%	12%	6%	12%	6%	18%	18%	0%

For use in creating the final regulate discharge-frequency curve at Castle Rock, the individual storm events were consolidated into a composite curve representing the estimated median value using the Total Probability Theorem. Each pattern is assigned a probability according to the number of occurrences seen from the seventeen historical events. The individual and composite results are shown graphically in Figure B. 20. The composite results for the design AEPs shown in Table B. 5 were interpolated on a probability scale and rounded to the thousands.

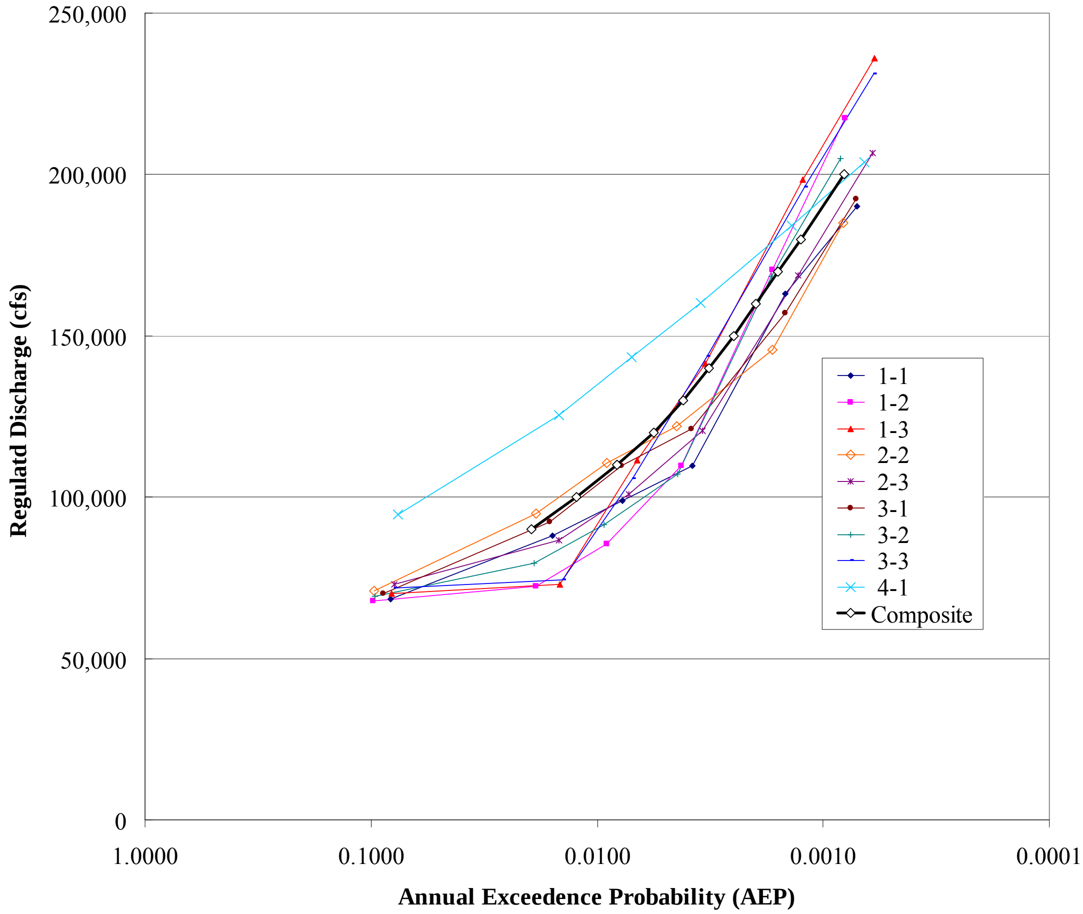


Figure B. 20. Storm patterns at various design events and the calculated composite curve.

Table B. 5. Composite discharge-frequency results for the synthetic analysis.

AEP	Discharge (cfs)
0.020	90,000
0.010	105,000
0.005	124,000
0.002	160,000
0.001	190,000

B.5. REGULATED DISCHARGE-FREQUENCY CURVE AT CASTLE ROCK

Figure B. 21 shows the updated regulated discharge-frequency curve with 90-percent confidence limits (discussed in section B.7). Also shown are the existing regulated discharge-frequency curve from the 1997 study and the unregulated peak discharge-frequency curve from the current analysis. The updated regulated curve is graphically drawn, first through the 83 measured and estimated annual maximum regulated discharges, and then using the composite results from the synthetic analysis beyond that. The measured and estimated points are plotted using Wiebul plotting position and a historical period of 114 years. Tabulated results are shown in Table B. 6.

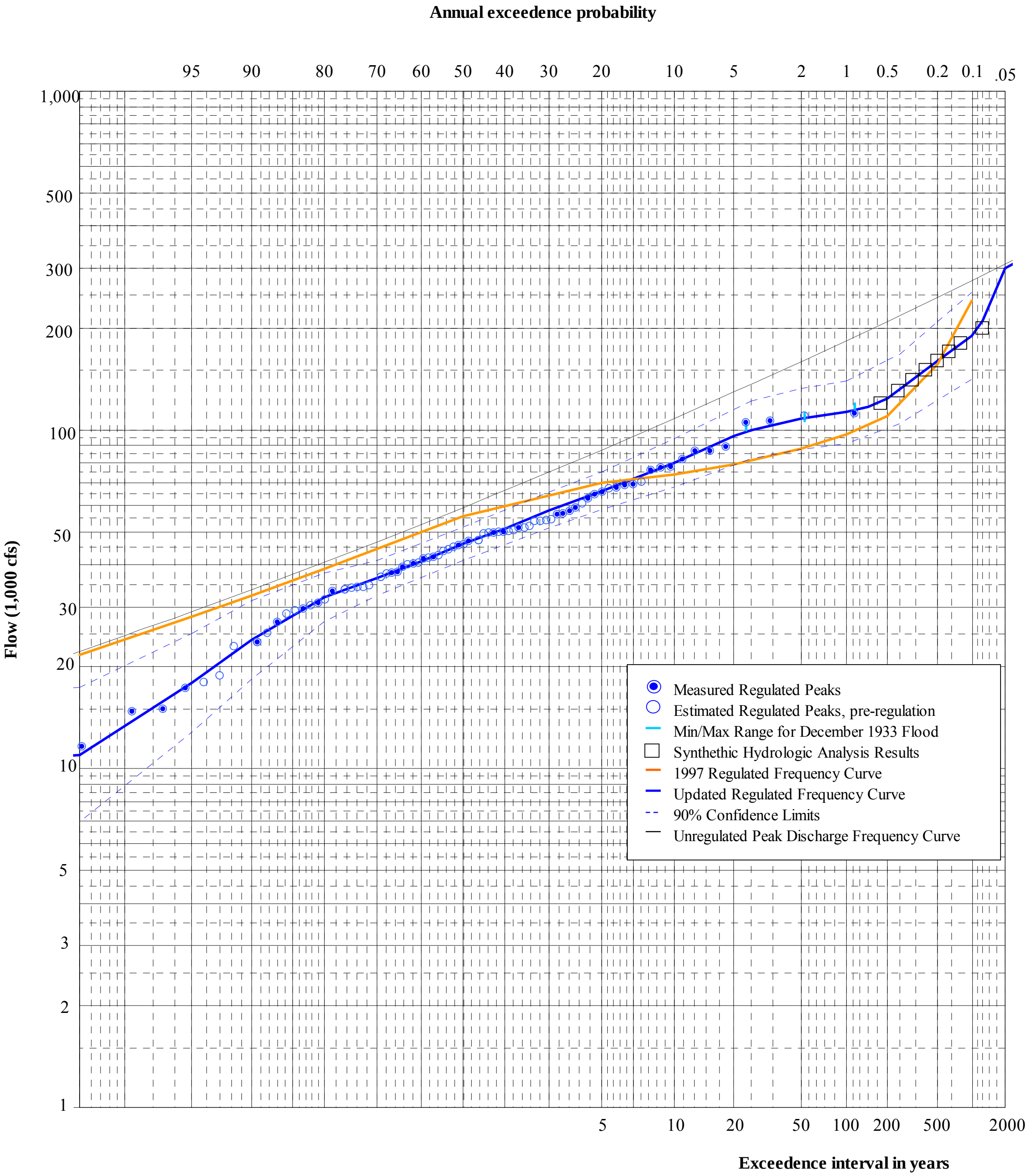


Figure B. 21. Updated regulated peak discharge-frequency curve at Castle Rock.

Table B. 6. Cowlitz River at Castle Rock peak regulated discharge-frequency curve.

AEP (%)	Discharge (cfs)
99	11,000
95	18,000
90	24,000
80	32,000
70	36,500
60	41,000
50	46,000
40	51,000
30	58,000
20	66,000
10	80,000
5	96,000
4	100,000
2	108,000
1	113,000
0.7	117,000
0.5	124,000
0.2	160,000
0.1	195,000
0.08	210,000
0.05	300,000
0.01	390,000

B.6. REGULATED DISCHARGE-FREQUENCY CURVES BELOW CASTLE ROCK

In the 1997 report, discharges downstream of Castle Rock were incrementally increased at Arkansas Creek (RM 16.50), Ostrander Creek (RM 8.54), and Coweeman River (RM 1.70) to account for additional local flow below Castle Rock. The flow adjustments were calculated using drainage area ratios applied to the natural discharge at Castle Rock. By using drainage area ratios, the locals below Castle Rock are assumed to have similar rainfall intensities and have identical hydrographs shapes as the entire Cowlitz basin above Castle Rock. This results in an unrealistic overestimation of locals inputs below Castle Rock. For the current update, flow frequency statistics are estimated for the three largest tributaries below Castle Rock (Arkansas Creek, Ostrander Creek, and Coweeman River) and discharges are adjusted for differences in hydrograph shape.

There are no long-term, systematic records of peak flow available for Arkansas Creek and Ostrander Creek. Discharge frequency statistics are calculated for Arkansas Creek and Ostrander Creek using the USGS Streamstats (USGS StreamStats 2008) web-based tool. The results of this analysis are shown in the Table B. 7.

Table B. 7. Peak discharge frequency data for Arkansas and Ostrander Creeks.

AEP	Arkansas Creek Drainage Area = 44.7 sq. mi. (cfs)	Ostrander Creek Drainage Area = 25.8 sq. mi. (cfs)
99.9	410	230
99	580	325
95	790	440
90	930	520
80	1,130	620
70	1,300	710
60	1,470	790
54	1,570	820
50	1,620	881
40	1,800	960
30	2,040	1,100
20	2,300	1,220
10	2,740	1,470
5	3,200	1,700
2	3,770	2,010
1	4,220	2,240
0.5	4,650	2,480
0.2	5,310	2,790
0.1	5,700	3,010

Source: USGS StreamStats

Discharge-frequency data for Coweeman River were estimated using the 35 years of peak discharge data from 1950 through 1984, available for the USGS gage Coweman (Coweeman) River near Kelso (USGS station #14245000). The peak discharge from the February 1996 flood is also available for this gage and it was used in the analysis as a historic peak flow. The peak flow frequency data for Coweeman River is summarized in the Table B. 8.

Table B. 8. Peak discharge-frequency data for Coweeman River at the gage and adjusted to the confluence with the Cowlitz River.

AEP	Coweeman River Drainage Area = 119 sq. mi. (cfs)	Coweeman River at Cowlitz River Drainage Area = 127 sq. mi. (cfs)
99.9	1,360	1,420
99	1,930	2,000
95	2,600	2,690
90	3,020	3,140
80	3,620	3,760
70	4,110	4,270
60	4,580	4,750
54	4,850	5,040
50	5,040	5,240
40	5,550	5,760
30	6,140	6,380
20	6,900	7,160
10	8,070	8,380
5	9,150	9,500
2	10,500	10,900
1	11,500	11,900
0.5	12,400	12,900
0.2	13,700	14,200
0.1	14,600	15,200

Short interval gage data are not available at these sites to accurately assess the timing of the peaks on the tributaries relative to the peak flow on the Cowlitz River; however, based on their relative size, it is assumed they would have a shorter time-to-peak than the Cowlitz at Castle Rock. To confirm this, discharge data for the nearby East Fork Lewis River near Heisson, WA gage (USGS 14222500) were used to estimate the timing of the Cowlitz tributary peaks relative to the Cowlitz River peak flow. The East Fork Lewis River Basin is about 25 miles south-southeast of the Coweeman River Basin. The drainage area at the East Fork Lewis gage is 125 square miles, similar to the Coweeman River Basin. Data for 17 events from water years 1996 through 2007 were available for analysis. Peak discharge on the East Fork Lewis occurred from 1 to 12 hours before the peak flow on the Cowlitz River at Castle Rock. At the time of the Castle Rock peak flow, the flow on the East Fork Lewis ranged from 56% to 98% of its peak flow with an average of 80%.

Based on this information, 80% of the peak discharge at each tributary is added to the Cowlitz River peak discharge at Castle Rock to estimate the discharge-frequency data for the modeled flow-change locations on the Cowlitz River at Arkansas Creek, Ostrander Creek and Coweeman River. For example, the 0.01 AEP Cowlitz River peak discharge at Ostrander Creek is the 0.01 AEP discharge at Castle Rock plus 80% of the 0.01 AEP discharge for Arkansas Creek plus 80% of the 0.01 AEP discharge at Ostrander Creek. The final discharge-frequency data for the Cowlitz River below Castle Rock to the Columbia River confluence are summarized the Table B. 9.

Table B. 9. Cowlitz River peak regulated discharges below Castle Rock to the Columbia River confluence (cfs).

AEP	below Arkansas Creek	below Ostrander Creek	below Coweeman River
	RM 16.5	RM 8.54	RM 1.7
99	11,500	11,700	13,300
95	18,600	19,000	21,100
90	24,700	25,200	27,700
80	32,900	33,400	36,400
70	37,500	38,100	41,500
60	42,200	42,800	46,600
50	47,300	48,000	52,200
40	52,400	53,200	57,800
30	59,600	60,500	65,600
20	67,800	68,800	74,500
10	82,200	83,400	90,100
5	98,600	99,900	107,500
4	102,700	104,100	112,000
2	111,000	112,600	121,300
1	116,400	118,200	127,700
0.7	120,600	122,400	132,400
0.5	127,700	129,700	140,000
0.2	164,200	166,500	177,800
0.1	199,600	202,000	214,100
0.08	214,700	217,100	229,500
0.05	304,900	307,500	320,400
0.01	395,800	398,800	413,500

The method used for calculating locals below Castle Rocks assumes negligible peak attenuation and coincident frequency. Also, the 80% reduction in peak essentially due to time-to-peak and basin size likely overestimates the discharges at the smaller tributaries of Arkansas and Ostrander Creeks, especially for extremely large events where the regulated peak discharge at Castle Rock would not occur with the locals but after the reservoir fills, resulting in a later surge well after the other larger tributaries have peaked. The fact that the three tributaries account for only 83% of the area below Castle Rock counteracts the previous factors to some degree; however, the estimates are still considered to be slightly conservative. Despite the relatively coarse methods used here, the updated estimates of local flow below Castle Rock are improvements upon the methods used in the 1997 study, which used the drainage area method applied to the Cowlitz River at Castle Rock to estimate local flows. The current method considers basin-specific hydrology for the local tributaries and adjusts for coincident peak timing.

B.7. HYDROLOGIC UNCERTAINTY

Hydrologic uncertainty for the regulated discharge-frequency curve at Castle Rock is estimated based the systematic record length adjusted for additional uncertainty for years where the regulated peak was estimated. In this study, 42 years were measured data, 42 years were estimated using a regulated-unregulated relationship, and one year was estimated using a detailed reservoir routing.

Uncertainty due to these methods is assessed graphically by assuming minimum and maximum conditions. For the WY 1934 event, the range of possible regulated discharges from the detailed routing analysis is depicted graphically at the plotting positions potentially affected, which in this case are the largest four events, plotted from 0.043 to 0.0087. One standard deviation for the graphical frequency curve through this range is approximately 4,000 cfs,

The uncertainty due to using a regulated-unregulated relationship is addressed by plotting minimum and maximum frequency curves based on upper and lower bounds of regulated-relationship as shown in figure B10. The difference between the regulated frequency curves is considerable at more frequent events; however, the curves are identical for events with AEPs less than 0.08. Because the LOP is concerned primarily with discharges in the 0.01 to 0.005 range, it is assumed that the hydrologic uncertainty due to the application of the regulated-unregulated relationship is negligible.

In FDA, an equivalent years-of-record (EYR) is used to describe hydrologic uncertainty. For the current update, the EYR used in FDA is back-calculated to include the uncertainty due to the use of the estimated regulated data. For the purposes of the LOP update, the critical discharge frequency chosen for the uncertainty analysis EYR back-calculation is 0.01 AEP (1/100). An EYR of 75 years effectively produces the uncertainty of an 83 EYR with an additional standard deviation of 4,000 cfs.

The additional 30 years of historical record beyond the systematic record increases certainty but at a significantly reduced value, estimated at 50% for this analysis. An additional 15 years of EYR are added to the 75 years representing the observed period of record for a total of 90 years representing the EYR of this hydrologic study. The 90 EYR at Castle Rock is applied for the entire reach including the three flow-change locations below Castle Rock.

B.8. ATTACHMENTS

B.8.1. Unregulated Discharge-Frequency Curves

B.8.1.1. Cowlitz River at Castle Rock
B.8.1.2. Cowlitz River near Randle (Upper Cowlitz)
B.8.1.3. Tilton River
B.8.1.4. Toutle River
B.8.1.5. Castle Rock Locals
B.8.1.6. Reservoir Locals

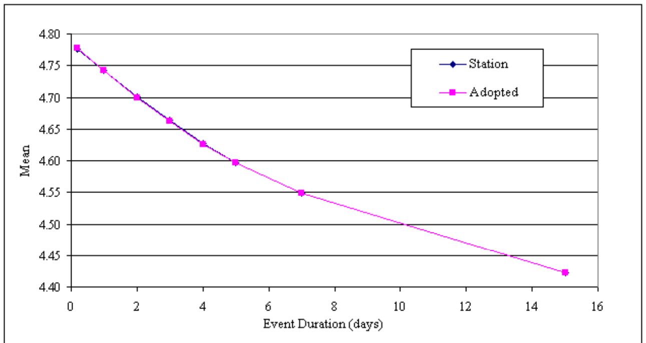
B.8.2. Historical Storm Patterns

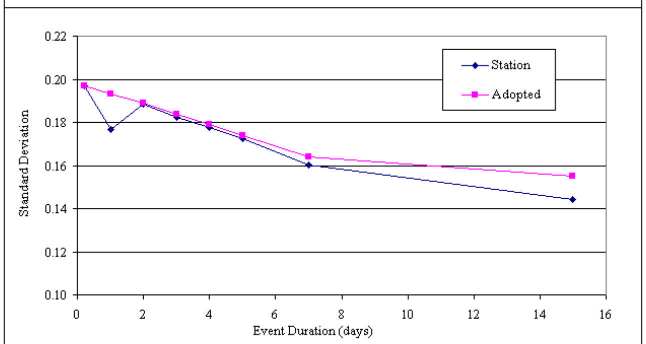
B.8.3. Synthetic Storm Centerings

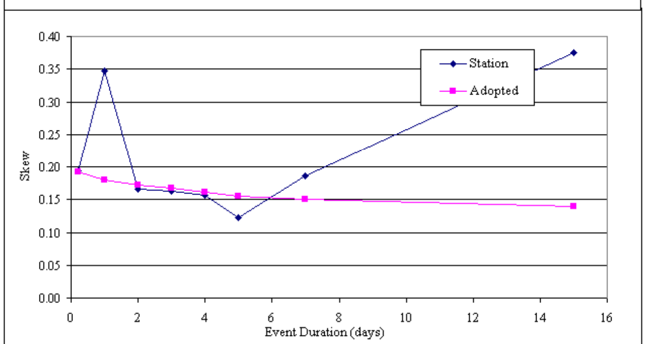
B.8.3.1. 0.10 AEP Centerings
B.8.3.2. 0.20 AEP Centerings
B.8.3.3. 0.01 AEP Centerings
B.8.3.4. 0.005 AEP Centerings
B.8.3.5. 0.001 AEP Centerings

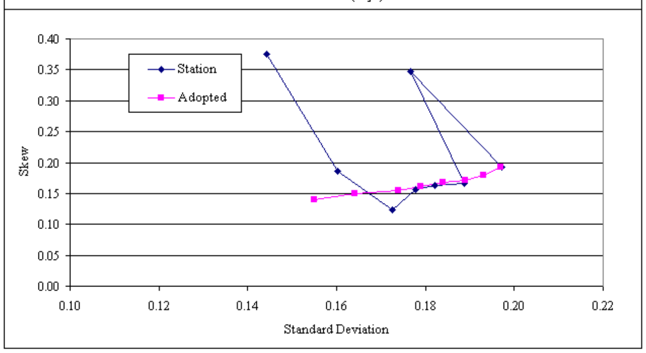
Cowlitz River at Castle Rock

	0.20	1	2	3	4	5	7	15
Mean	4.777	4.743	4.700	4.663	4.627	4.598	4.549	4.423
Standard Deviation	0.197	0.177	0.189	0.182	0.178	0.173	0.160	0.144
Station Skew	0.193	0.347	0.166	0.163	0.157	0.123	0.185	0.376
N	69	74	79	79	79	79	78	76
H or "n"	114	114	83	83	83	83	83	83
L	1	7	2	2	2	2	3	4
Z	1	1	1	1	1	1	1	1
Low-outlier threshold		25,000	19,000	18,000	17,000	15,000	15,000	12,000
Adopted Mean	4.777	4.742	4.700	4.663	4.626	4.597	4.549	4.423
Adopted Standard Deviation	0.197	0.193	0.189	0.184	0.179	0.174	0.164	0.155
Adopted Skew	0.193	0.180	0.172	0.168	0.161	0.155	0.150	0.140





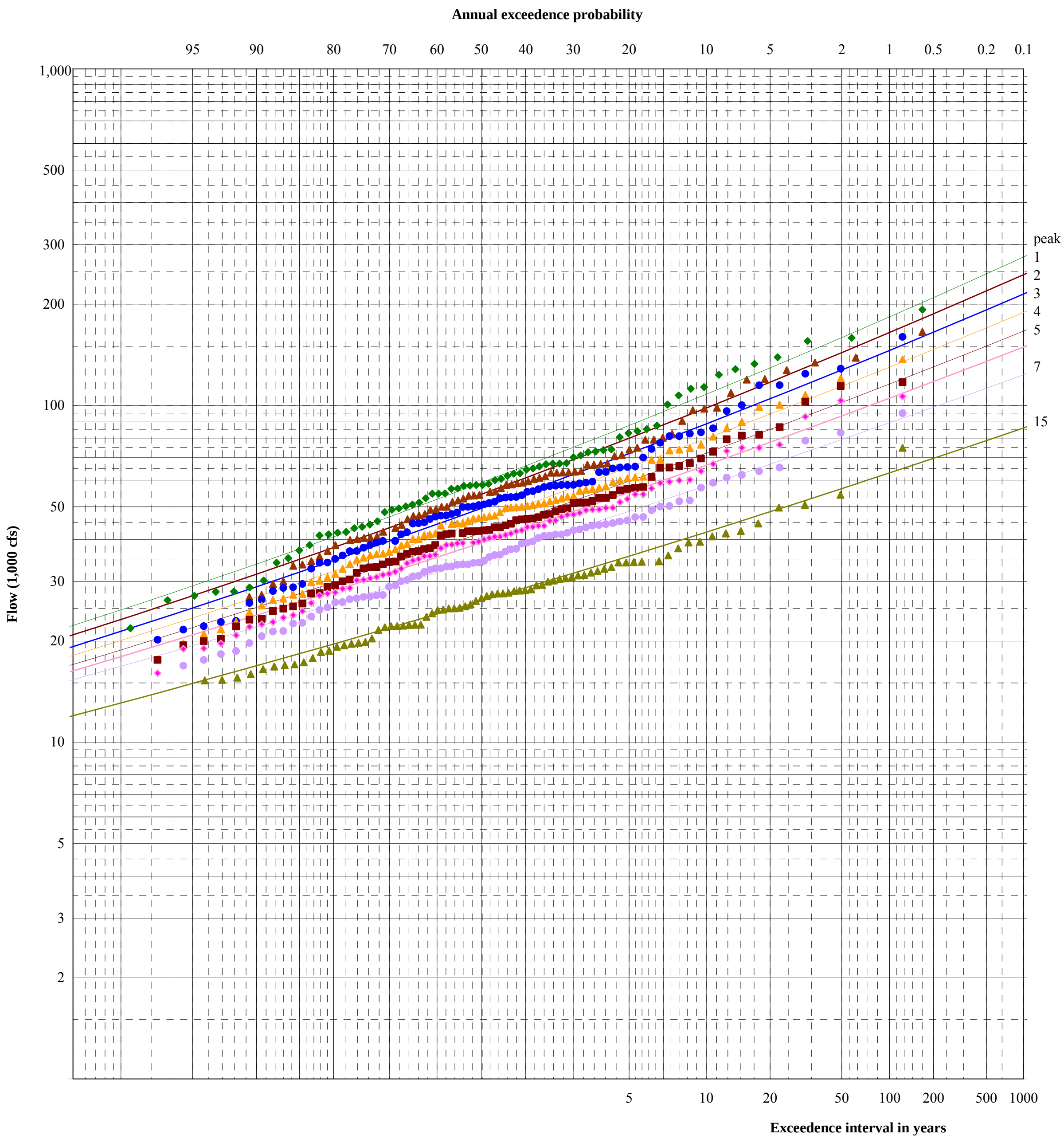




	Peak	1	2	3	4	5	7	15
1927	41,100	40,800	36,900	35,367	33,025	30,920	27,057	19,580
1928	74,000	71,300	65,000	56,933	51,500	47,940	41,871	32,727
1929	21,800	21,100	20,150	19,633	19,375	18,980	18,271	16,467
1930	30,200	29,500	28,100	26,700	25,300	23,840	22,514	21,527
1931	65,900	63,200	56,400	51,200	45,600	40,760	34,057	25,053
1932	50,800	49,200	48,100	44,467	42,300	39,140	33,700	28,247
1933	53,000	52,200	46,750	44,467	44,025	43,900	41,371	30,207
1934	139,000	134,000	128,500	120,333	114,125	106,300	94,814	74,873
1935	61,800	58,100	58,100	53,800	49,175	45,540	38,871	29,201
1936	48,200	39,800	39,600	35,833	33,700	31,400	27,343	24,760
1937	49,000	47,200	42,000	40,167	36,775	34,640	30,314	22,320
1938	62,900	60,500	57,600	53,567	51,475	47,440	40,986	26,763
1939	43,600	39,800	34,100	29,867	27,700	27,840	25,129	16,945
1940	41,900	40,500	39,600	36,767	33,100	31,660	30,057	22,333
1941	26,400	22,100	22,100	19,633	17,550	16,038	14,239	11,810
1942	58,100	55,400	50,850	47,067	42,550	39,400	34,171	23,543
1943	60,500	58,300	51,300	46,867	42,325	39,020	33,629	25,667
1944	37,100	33,700	29,500	26,467	23,225	20,760	17,543	11,420
1945	45,300	44,200	41,400	36,333	32,825	30,340	27,271	19,373
1946	54,700	51,700	50,150	45,433	41,450	38,940	35,914	29,967
1947	85,100	82,400	81,050	80,767	79,350	74,880	65,400	42,347
1948	38,500	37,000	34,150	32,700	30,400	28,560	26,586	22,287
1949	51,500	43,300	35,850	30,167	27,625	25,900	26,057	22,107
1950	54,800	54,000	51,750	49,933	47,550	43,920	37,386	34,433
1951	67,200	63,900	63,300	61,467	57,250	54,440	46,643	30,547
1952	41,400	35,600	35,000	31,600	28,900	28,680	26,000	19,726
1953	60,000	50,000	46,000	44,000	41,750	40,400	39,143	35,867
1954	73,000	63,600	55,650	51,067	48,425	45,440	39,443	31,260
1955	50,100	43,300	39,150	33,800	30,050	27,620	23,614	15,921
1956	67,400	63,100	57,200	49,733	45,825	42,260	37,500	34,160
1957	43,200	40,400	38,500	37,400	36,225	34,980	32,857	27,467
1958	35,200	34,500	32,750	30,867	29,200	27,280	24,700	18,687
1959	57,800	52,900	49,900	44,733	41,750	39,140	34,514	27,653
1960	62,700	61,000	58,650	56,433	52,075	49,620	43,943	28,087
1961	64,500	59,000	50,550	46,133	41,650	43,520	42,600	31,880
1962	42,100	40,100	36,950	34,833	31,825	30,240	29,171	24,133
1963	66,900	59,500	57,750	52,900	46,525	41,640	40,386	32,233
1964	72,600	55,400	53,050	46,267	41,100	37,680	33,243	26,260
1965	83,900	79,100	74,300	68,933	65,400	60,120	52,243	34,433
1966	28,800	27,000	25,900	24,333	23,100	22,400	20,657	16,780
1967	65,000	58,800	53,400	48,167	42,700	38,520	33,200	25,313
1968	49,500	47,600	47,050	44,633	43,425	42,600	41,086	28,193
1969	58,514	55,527	53,324	50,042	43,419	41,234	36,834	24,673
1970	56,686	54,198	53,478	50,294	47,341	47,316	44,149	37,652
1971	54,667	48,988	44,735	41,403	37,683	35,725	36,014	31,351
1972	107,187	96,847	82,468	73,925	66,048	58,867	48,838	40,833
1973	73,732	66,683	58,964	55,771	54,183	49,695	46,603	34,146
1974	113,411	108,987	100,195	89,393	81,277	76,471	63,708	44,599
1975		61,567	58,047	50,645	46,123	46,685	44,726	34,263
1976	128,096	119,361	114,795	107,414	102,603	92,637	78,577	49,770
1977		13,929	13,617	12,799	11,848	11,028	10,280	8,477
1978	158,796	127,335	114,945	100,410	81,809	73,267	62,200	54,273
1979		33,380	28,831	27,567	25,800	24,520	22,598	17,768
1980		38,433	37,816	36,187	34,360	32,757	31,081	22,121
1981								
1982		67,034	65,654	60,803	56,904	56,687	56,967	41,705
1983		63,322	54,081	52,340	51,327	49,045	45,554	29,320
1984		46,993	44,948	41,198	38,466	35,717	31,177	24,962
1985		23,444	22,723	21,641	19,954	18,932	16,868	15,556
1986		70,690	65,382	59,186	53,118	49,108	41,378	27,774
1987		79,128	70,021	59,885	53,153	48,862	42,827	28,434
1988	57,724	53,948	47,587	38,844	34,279	30,650	26,756	18,516
1989	28,000	21,970	21,576	20,928	20,268	19,637	18,632	15,258
1990	123,357	89,995	83,208	74,646	65,290	59,342	50,066	30,634
1991	100,700	98,503	85,510	76,500	67,375	59,854	50,276	33,085
1992	44,284	42,181	39,582	38,373	37,429	36,039	32,026	22,184
1993	34,104	30,058	28,731	27,415	24,900	22,723	19,692	15,301
1994	27,941	27,348	26,471	25,477	24,501	23,413	21,371	17,003
1995	67,172	56,806	49,968	46,563	42,368	38,607	32,517	27,139
1996	192,618	165,455	159,728	137,099	117,318	103,445	82,900	50,676
1997	87,107	80,200	77,441	73,567	69,554	67,041	58,925	39,355
1998	27,211	23,097	22,911	22,786	22,051	22,005	21,297	19,831
1999	70,181	67,276	59,346	57,475	55,960	51,624	43,423	27,618
2000	80,525	74,885	63,484	56,216	49,497	43,571	35,476	24,956
2001	13,689	13,219	12,050	10,982	10,310	9,721	8,753	6,656
2002	71,093	66,716	58,051	49,577	44,510	39,970	33,724	21,929
2003	112,077	97,842	81,021	68,911	61,292	54,440	44,395	30,902
2004	56,559	49,996	47,150	41,629	36,895	33,980	28,930	20,329
2005	58,006	45,946	44,580	40,109	35,565	32,075	26,995	17,268
2006	82,753	73,960	65,849	61,274	56,576	52,738	45,209	34,340
2007	155,352	138,606	124,143	98,995	86,204	74,893	61,038	39,148
2008		63156	55381	51904	46017	40746	32830	19188
2009	132,849	119,656	96,200	85,564	72,863	63,808	51,928	

Notes:
All 1- through 15-day duration data pre-1969 are adjusted as described in report
Various WY's and durations from 1969 - 2009 are calculated from hourly hydrographs
All other WY's and durations from 1969-2009 are from daily modeled data
WY1934 1- through 15-day duration max are calculated from synthetic hourly data
WY 1989 and 1991 peaks are from 1997 study

Cowlitz River at Castle Rock



Notes:
Period of Record for Regulated Flows Water Years 1927-2009.
Data from WYs 1981-1985 are not included in the analysis, these five years are part of the historical period.
Historic period for peak and 1-day curves is 1896-2009.
Historic period for 2- through 15-day curves is from 1927-2009.
Data are plotted using median plotting positins

COWLITZ RIVER STUDY
COLUMBIA RIVER BASIN

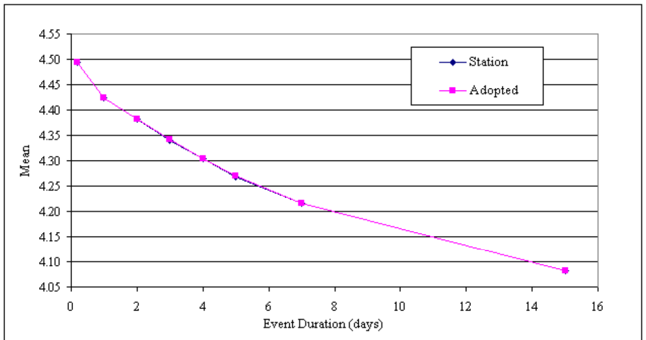
VOLUME DURATION FREQUENCY CURVES

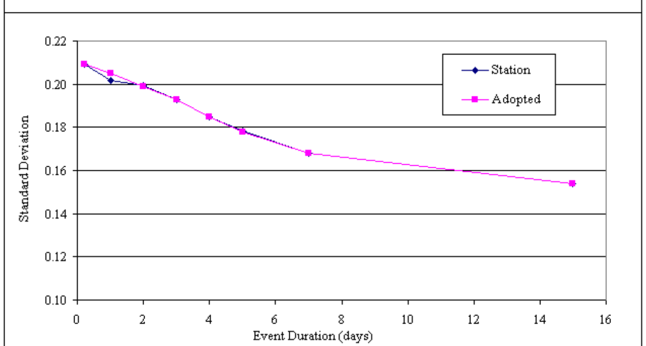
Cowlitz River at Castle Rock

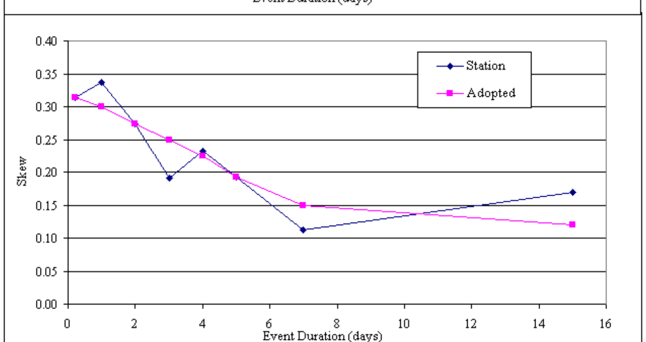
U.S ARMY CORPS OF ENGINEERS
Portland District, EC-HY

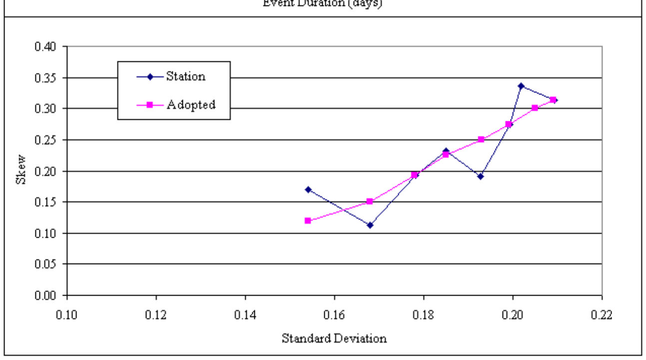
Cowlitz River near Randle

	0.20	1	2	3	4	5	7	15
Mean	4.494	4.425	4.382	4.341	4.303	4.269	4.215	4.082
Standard Deviation	0.209	0.202	0.199	0.193	0.185	0.178	0.168	0.154
Station Skew	0.314	0.337	0.274	0.191	0.232	0.192	0.113	0.170
N	70	69	69	69	68	68	68	69
H or "n"	70	71	71	71	71	71	71	71
L	0	2	2	2	3	3	3	2
Z	0	0	0	0	0	0	0	0
Low-outlier threshold		8,000	8,000	8,000	8,000	8,000	6,000	
Adopted Mean	4.494	4.425	4.382	4.341	4.303	4.269	4.215	4.082
Adopted Standard Deviation	0.209	0.205	0.199	0.193	0.185	0.178	0.168	0.154
Adopted Skew	0.314	0.300	0.274	0.250	0.225	0.192	0.150	0.120





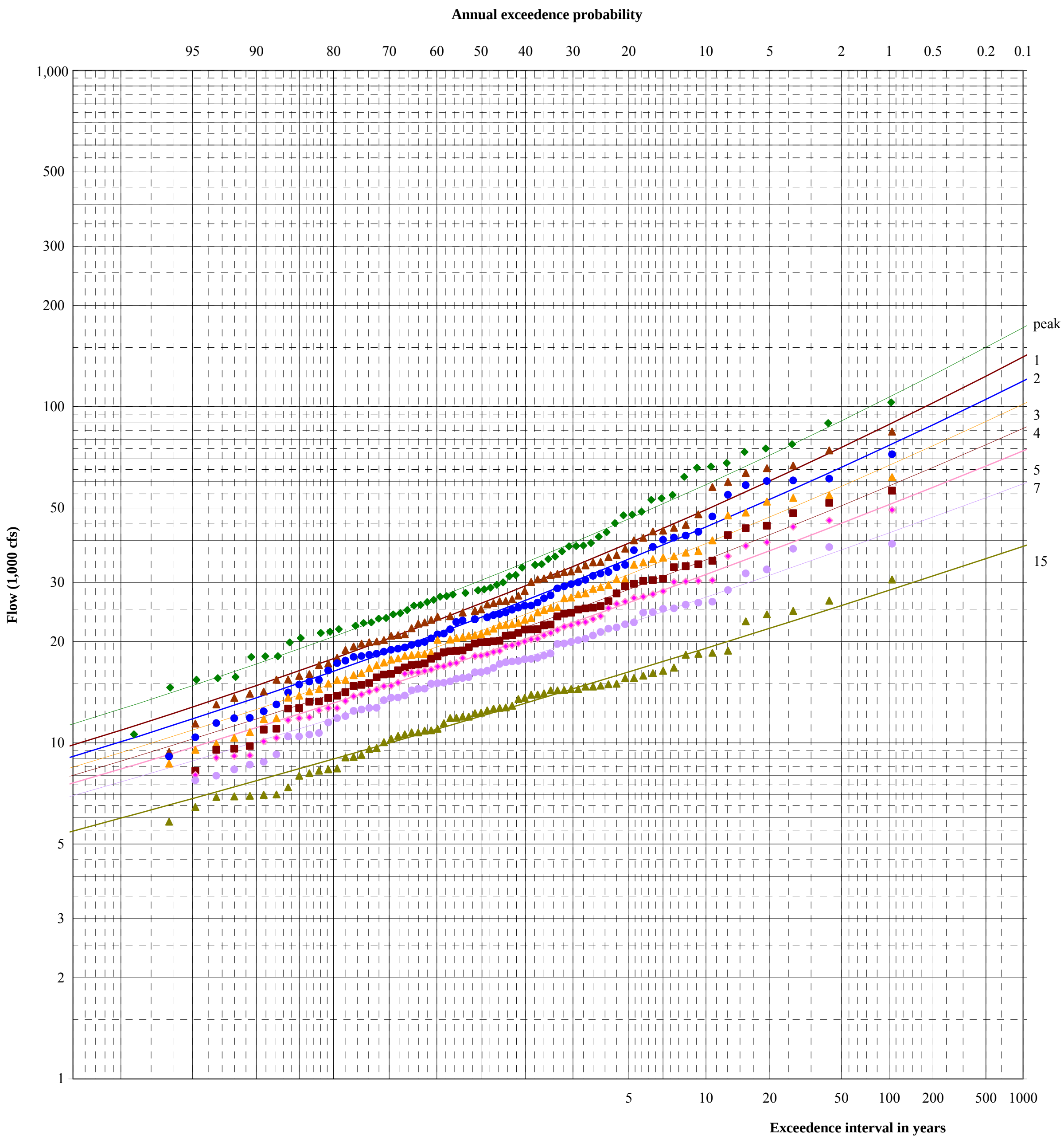




	Peak	1	2	3	4	5	7	15
1927	19875	19333	17541	15881	15048	14195	12694	10281
1928	31565	26371	25558	22742	20798	19728	17751	14698
1929	15558	15413	14883	14451	14139	13918	13619	12491
1930	18076	17016	15370	14217	13231	12674	11999	10040
1931	28418	24945	21129	17837	16042	14731	12702	9647
1932	28418	26014	25248	22566	19890	18155	15580	11955
1934	75092	63521	60994	53511	48169	43882	37753	30584
1935	28957	26727	26134	24435	21728	19503	16693	11956
1947		35636	33221	30653	29187	27664	24369	16370
1948	21763	20000	18000	16667	14750	13300	11486	9040
1949	24100	13000	11850	10753	9773	9130	8327	6893
1950	27600	23800	23600	21067	18750	17280	15657	12706
1951	33800	32300	31350	30667	27800	25900	22029	13894
1952	18000	16000	15200	13800	12670	11828	10574	7979
1953	25600	24400	21750	20233	18725	18100	17500	16112
1954	24800	22800	19200	18133	17000	16060	14471	12735
1955	24300	14200	13000	11760	10935	10090	8770	6923
1956	33200	30600	28800	24833	21700	19360	16367	14989
1957	22300	20900	20450	20300	19725	18620	17443	14313
1958	20500	18900	18100	17000	15650	14394	12554	9074
1959	23500	21900	19750	18533	17225	16360	15486	13500
1960	47500	40700	38250	34400	30450	28220	24457	14929
1961	22700	20800	18400	17333	15950	14780	13379	11866
1962	22800	17200	16400	15000	13575	12676	12426	10865
1963	41000	34400	33800	28700	24350	21216	19740	13902
1964	18100	11400	10385	9930	9523	9166	8601	7017
1965	45000	42500	40750	37167	33975	30300	25100	15837
1966	21400	19900	19050	17667	16750	16180	15029	11005
1967	29500	23800	22850	20300	18075	16500	14457	10477
1968	27900	25800	24150	23233	22350	22140	21343	14700
1969	26600	23200	21000	20500	20025	18760	16206	10890
1970	28600	26400	23600	21433	20875	20860	19671	14446
1971	26200	22600	19950	18233	16425	15098	13610	11849
1972	34000	32200	27450	25300	23750	22740	20843	18800
1973	35200	31800	29200	26900	24875	22840	20186	14437
1974	65800	59700	54650	48400	43475	39480	32786	22987
1975	27200	24400	23100	20833	19225	17860	17157	14343
1976	73300	66800	60050	54500	51650	45840	38214	24095
1977	9800	9390	9105	8673	8248	8014	7731	7000
1978	89300	74100	60300	52200	44200	38540	31900	26453
1979	15400	15400	14100	13633	12638	11680	10440	8119
1980	22300	21000	19550	18367	17075	16200	14986	10594
1981	52800	42800	42400	36800	33525	30100	26271	18271
1982	38400	33700	30500	27833	25475	25160	25743	18391
1983	30000	27400	23300	22333	21200	20000	17886	12288
1984	25700	24700	23950	21833	19925	18340	16200	10736
1985	27900	7830	7755	7370	6873	6496	5960	5821
1986	33800	32900	31850	29433	26375	23800	20400	14317
1987	31300	15800	11440	9527	7978	7038	5804	3705
1988	27300	23700	18650	15373	13260	11910	10437	7366
1989	15700	14000	11825	10350	9598	9026	7977	6435
1990	47700	43600	37400	33900	29700	26980	22800	14028
1991	66300	57600	47100	40000	34775	30420	25014	15578
1992	21200	19700	18900	18333	17775	17140	15229	10740
1993	21200	17900	17250	15400	13788	12488	10686	8259
1994	14600	13600	12400	11833	11010	10340	9226	6959
1995	68000	31500	26900	25167	22450	20300	17361	12889
1996	103000	84300	72150	61700	56200	49280	39071	24661
1997	54600	40000	38250	35867	33275	30180	26100	16725
1998	38500	30100	24400	22433	20100	18100	15074	9579
1999	39300	34500	29650	27633	25100	23320	19943	12638
2000	42300	37800	32200	29033	25225	22360	18421	12242
2001	10600	4350	4100	3830	3623	3446	3166	2649
2002	48700	36200	30000	27033	24200	21640	18159	11446
2003	61800	44500	40150	35133	30300	26300	21850	15574
2004	23400	20200	18250	16100	14875	13746	11824	8337
2005	38600	24700	23300	20600	18575	16852	14314	9195
2006	35800	28300	25600	23467	21775	20380	17714	13590
2007	77300	65600	58350	47467	41475	35860	28443	18519
2008	37100	30800	24850	20900	18825	16874	13826	8383
2009	53400	47800	41350	35533	30750	27180	22514	14791

Notes:
Data from Cowlitz River at Mossyrock, correlated using overlapping period from 1948-1960.
Data from Cowlitz River near Kosmos, no conversion.

Cowlitz River near Randle



Notes:
Period of Record for Regulated Flows Water Years 1948-2009.
Data are plotted using median plotting positins

COWLITZ RIVER STUDY
COLUMBIA RIVER BASIN

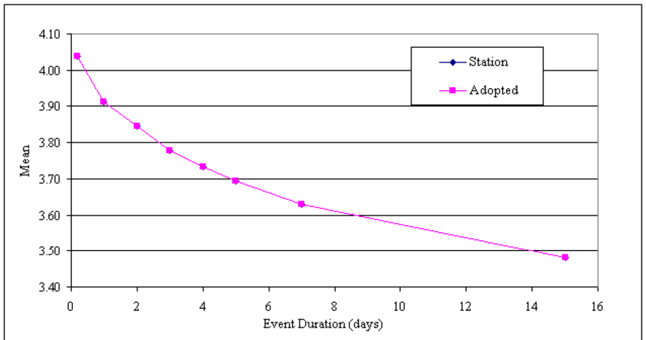
VOLUME DURATION FREQUENCY CURVES

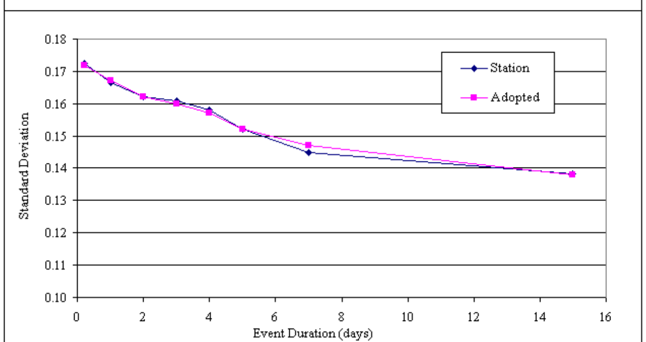
Cowlitz River near Randle

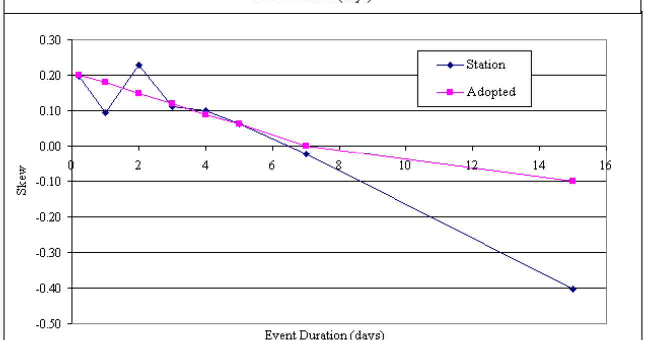
U.S ARMY CORPS OF ENGINEERS
Portland District, EC-HY

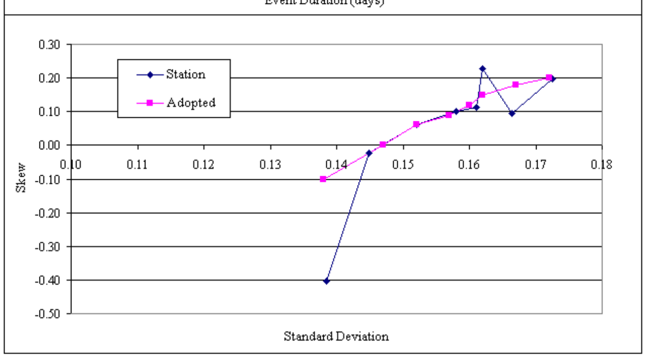
Tilton River above Bear Canyon

	0.20	1	2	3	4	5	7	15
Mean	4.038	3.913	3.846	3.779	3.733	3.693	3.630	3.482
Standard Deviation	0.173	0.166	0.162	0.161	0.158	0.152	0.145	0.138
Station Skew	0.198	0.095	0.228	0.112	0.100	0.063	-0.022	-0.404
N	61	64	63	66	66	66	66	67
H or "n"	68	68	68	68	68	68	68	68
L	7	4	5	2	2	2	2	1
Z	0	0	0	0	0	0	0	0
Low-outlier threshold	5,500	4,000	3,500	3,000	2,000			
Adopted Mean	4.038	3.913	3.846	3.779	3.733	3.693	3.630	3.482
Adopted Standard Deviation	0.172	0.167	0.162	0.160	0.157	0.152	0.147	0.138
Adopted Skew	0.200	0.180	0.150	0.120	0.090	0.063	0.000	-0.100





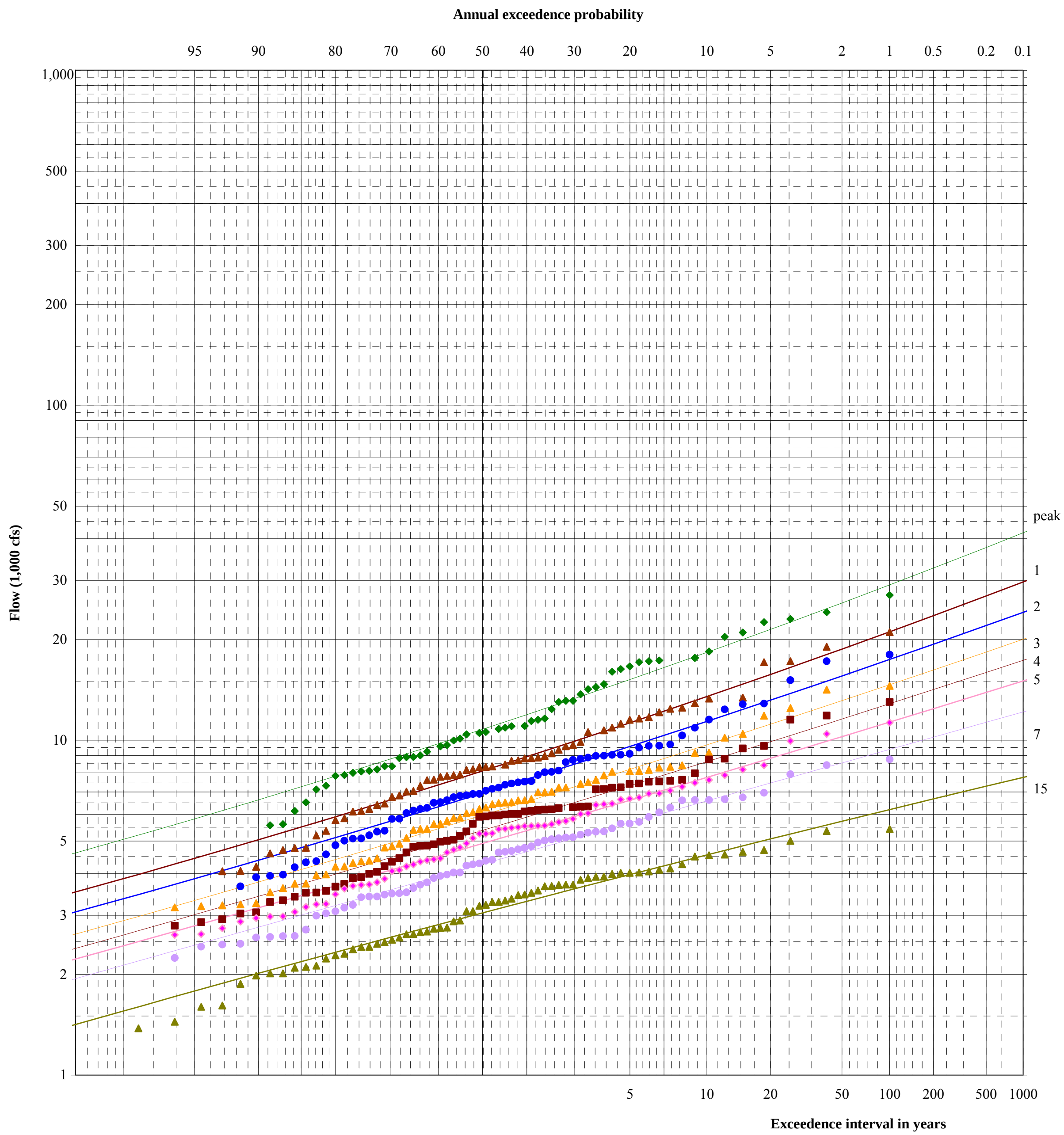




	Peak	1	2	3	4	5	7	15
1942	7312	6119	5197	4435	3900	3601	3082	2012
1943	8903	7050	5825	4775	4203	3764	3482	2535
1944	8180	6472	4854	3953	3408	2975	2417	1444
1945	7827	5848	4560	3974	3505	3176	2990	2495
1946	8361	6146	6056	5435	4833	4438	3968	3289
1947	13106	10575	8722	8074	7231	6688	5630	3831
1948	6146	5360	4316	3615	3326	3671	3410	2416
1949	6517	4763	4348	4197	3990	3697	3160	2669
1950	11027	8975	7863	6993	6151	5544	4619	4103
1951	11027	9129	8022	8074	7208	6466	5342	3286
1952	8840	6833	5328	4369	3873	3708	3455	2465
1953	10575	7827	6842	6613	6026	5613	5117	4550
1954	20337	12112	9612	7851	7398	6649	5699	4137
1955	12383	8189	6621	5622	4804	4196	3411	2228
1956	20969	9581	9490	7610	6322	5441	4362	3694
1957	10500	7890	6910	6047	5625	5256	4767	3669
1958	5610	4770	4175	3730	3500	3242	3040	2304
1959	13100	11600	8985	7407	6343	5558	4820	3703
1960	16000	11200	8870	8153	7530	6928	5896	3559
1961	11400	10700	8105	7227	6203	5538	4721	3704
1962	7870	6770	5365	4817	4330	4336	4024	3200
1963	16300	12400	9105	7483	6190	5268	4390	3321
1964	9980	8320	6230	5107	4440	4096	3616	2633
1965	13700	9680	9030	8413	7960	7464	6081	3665
1966	4850	4060	3405	3193	2918	2744	2466	2103
1967	9670	8840	6825	5743	4973	4408	3759	2876
1968	8900	7860	6280	5430	4888	4708	4221	2636
1969	8060	7020	6165	5853	5178	4622	3874	2411
1970	8100	7600	6905	6667	6023	5500	4944	4055
1971	9580	8460	7155	6433	5963	5476	4643	4023
1972	17600	13400	12350	10440	9440	8412	6750	4639
1973	10900	8350	7060	6293	5915	5550	5291	4015
1974	9580	7620	7360	6523	6003	5418	4664	3873
1975	13000	9870	9625	8100	7133	7266	6659	4491
1976	17600	12900	12850	12467	11840	10440	8424	5358
1977	3130	2650	2230	1947	1748	1622	1467	1379
1978	22500	17100	12805	10170	8758	7604	6610	5425
1979	11600	6410	5085	4300	4043	3854	3699	2575
1980	10500	8680	6530	6093	6125	5688	5016	3224
1981	10400	8140	7515	6617	6303	6042	5219	3449
1982	18400	11700	9705	8050	7608	7066	6963	4533
1983	11000	8310	8035	7407	7513	6936	6629	4015
1984	9240	7300	6495	5613	4838	4252	3489	2750
1985	4510	4070	3965	3513	3285	2978	2601	2122
1986	16600	8870	8595	7010	5905	5096	4023	2759
1987	17200	12500	10330	8327	7148	6406	5450	3461
1988	11500	5770	5010	4193	3545	3068	2599	1872
1989	4240	3600	3465	3260	3038	2872	2584	2093
1990	17300	13400	11515	9180	7560	6434	5093	3087
1991	17600	13300	10890	9150	8803	7856	6293	3983
1992	5290	4190	3935	3727	3650	3466	3221	2373
1993	5570	4700	3660	3210	2863	2622	2237	1613
1994	4450	3890	3415	3167	3060	2942	2717	1983
1995	10800	8820	6755	5393	4623	4064	3399	3087
1996	27100	21000	17200	14530	12990	11278	8767	5005
1997	10100	8460	7215	6493	6303	6016	5330	3915
1998	7990	4590	3895	3233	2793	2636	2451	2277
1999	17100	9360	8970	8280	7423	6740	5636	3359
2000	14700	10700	9045	7217	6213	5736	5061	3501
2001	3030	2100	2040	1843	1658	1506	1280	872
2002	8360	6230	5815	4910	5045	4918	4283	2735
2003	14200	11500	8805	6997	5958	5252	4257	2895
2004	10800	8710	7540	6230	5338	4784	3929	2681
2005	14400	10900	7475	5870	5003	4380	3513	2012
2006	9000	7770	7420	6513	6263	5824	5130	3908
2007	23000	17200	15100	11823	9608	8168	6626	4263
2008	7120	5190	5080	4347	3718	3236	2575	1599
2009	24100	19000	18000	14150	11520	9942	7910	4703

Notes:
Calculated using the drainage area ratio from the Tilton River near Cinebar gage

Tilton River above Bear Canyon



Notes:
Period of Record for Regulated Flows Water Years 1942-2009.
Data are plotted using median plotting positins

COWLITZ RIVER STUDY
COLUMBIA RIVER BASIN

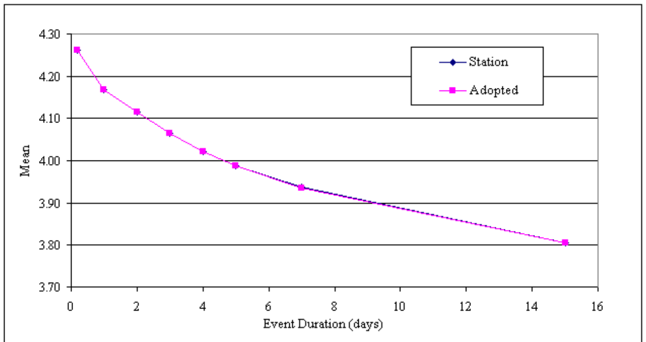
VOLUME DURATION FREQUENCY CURVES

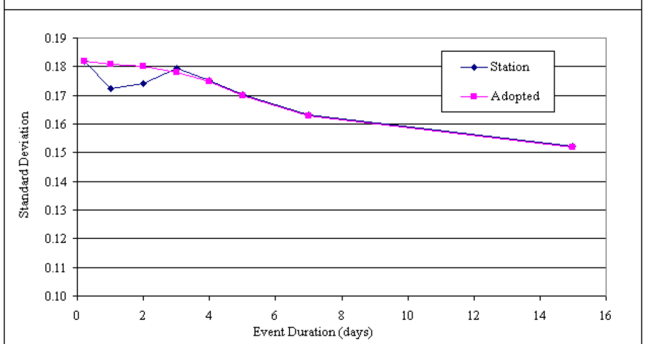
Tilton River above Bear Canyon

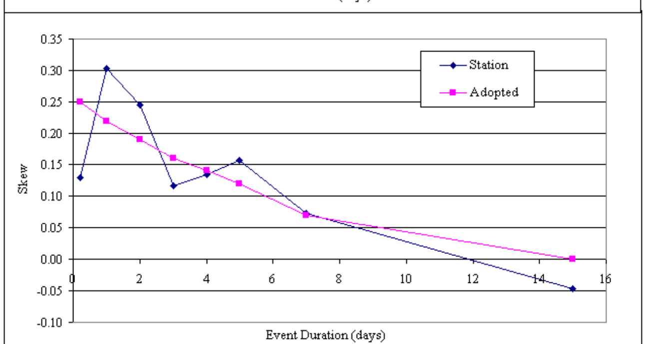
U.S ARMY CORPS OF ENGINEERS
Portland District, EC-HY

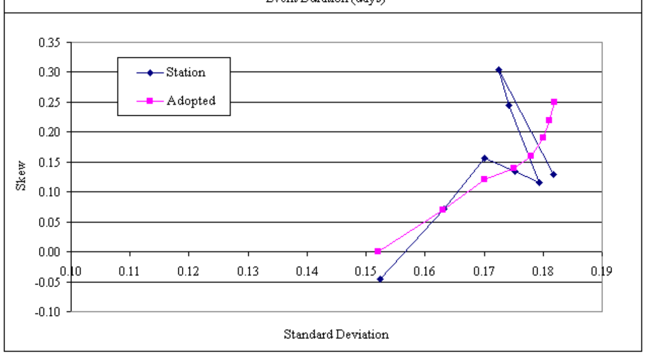
Toutle River near Silver Lake

	0.20	1	2	3	4	5	7	15
Mean	4.262	4.169	4.115	4.064	4.022	3.989	3.936	3.805
Standard Deviation	0.182	0.172	0.174	0.179	0.175	0.170	0.163	0.152
Station Skew	0.129	0.304	0.245	0.116	0.135	0.156	0.072	-0.046
N	78	76	76	77	77	77	77	77
H or "n"	87	84	84	84	84	84	84	84
L	2	2	2	1	1	1	1	1
Z	1	1	1	1	1	1	1	1
Low-outlier threshold	6,000	5,000	4,500					
Adopted Mean	4.262	4.169	4.115	4.064	4.022	3.989	3.936	3.805
Adopted Standard Deviation	0.182	0.181	0.180	0.178	0.175	0.170	0.163	0.152
Adopted Skew	0.250	0.220	0.190	0.160	0.140	0.120	0.070	0.000





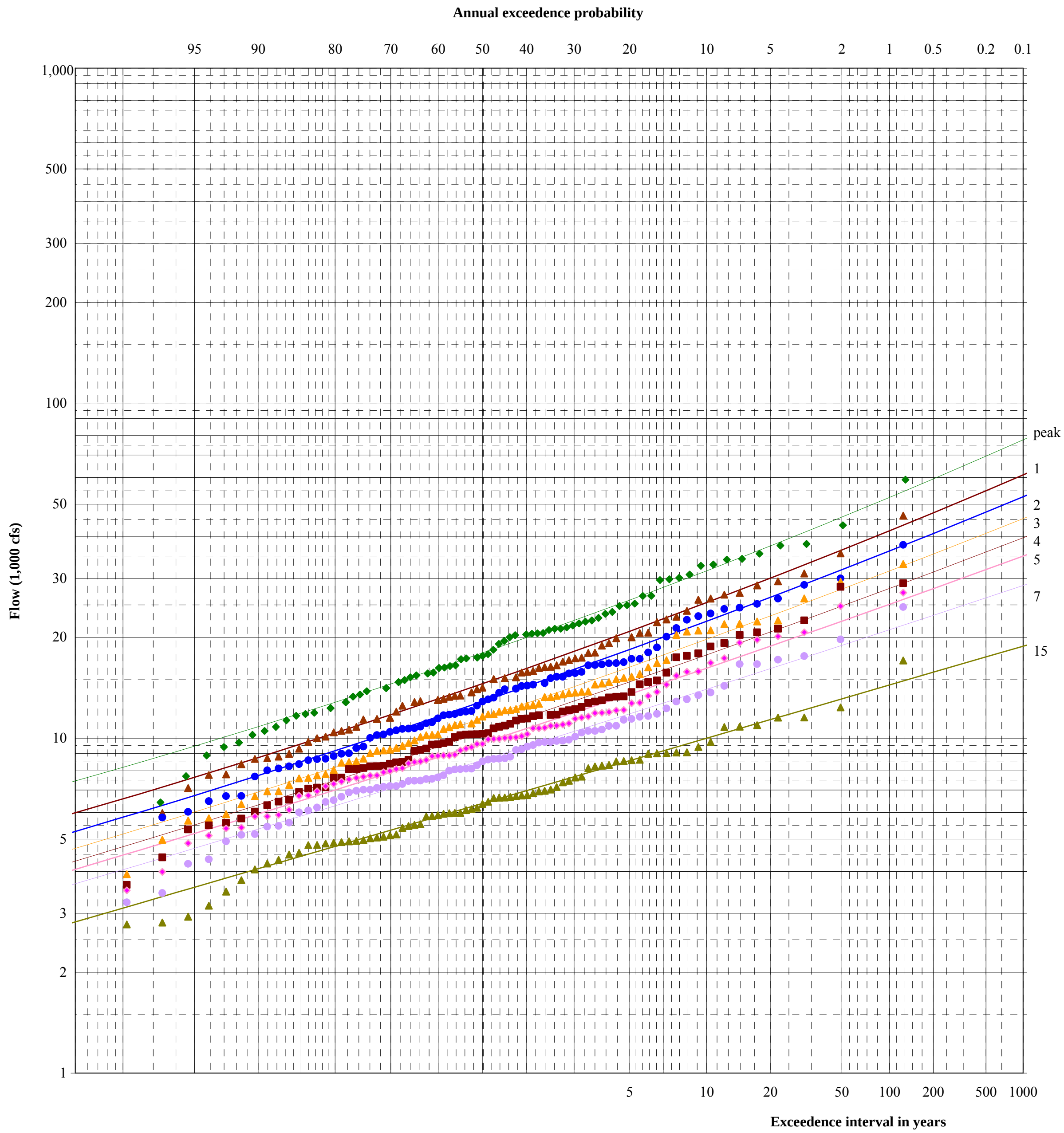




	Peak	1	2	3	4	5	7	15
1910	37600	35600	28650	26133	22450	20120	16634	10795
1911	23500	20600	16600	13510	11445	9940	8081	6935
1912	14700							
1920	9700	8810	7685	6940	6303	5904	5470	3769
1921	14700	13700	11700	11067	11275	10888	10877	8277
1922	13800							
1923	16500							
1930	10800	8360	6485	5763	5585	5408	5143	5083
1931	20000	15000	14600	12573	10898	9618	7994	5969
1932	15600	11400	10210	9017	8083	7424	6453	6099
1933	13300	13000	8835	7810	6905	7604	6961	4801
1934	34300	31000	30000	29333	29000	27200	24614	17058
1935	19500	15200	14450	13633	12210	10896	9431	7145
1936	15700	12000	11450	10653	9750	8876	7634	6844
1937	17400	11400	11150	10253	9565	8830	7553	5399
1938	20300	17100	16850	16267	14875	13370	11347	7021
1939	15400	13100	10440	9240	8240	7978	7016	4549
1940	12800	10800	10020	9187	8498	8858	8290	5892
1941	9410	7800	6025	4973	4405	3990	3450	2816
1942	21400	18900	17250	14693	12725	11514	9893	6457
1943	20500	15200	14050	11790	10078	9190	8627	5981
1944	12300	9300	8120	6707	5755	5118	4349	2930
1945	14900	10400	9445	8533	7618	6940	6209	4329
1946	17400	13000	12000	11067	10300	10040	9743	7629
1947	29800	22600	22550	22300	21200	20680	17571	10930
1948	14100	12500	11080	10093	9160	8484	7449	5979
1949	17300	12900	10550	9477	8470	7732	7749	6135
1950	17200	16500	15600	14467	12973	11886	9800	9061
1951	14100	13700	12500	12100	11725	10704	9217	6197
1952	11900	10500	9015	7743	7115	7182	6530	4962
1953	15600	14000	12850	12133	11725	11066	10603	9012
1954	25200	15700	14050	12430	11700	10744	9821	7378
1955	16200	13400	10820	9157	8140	7300	6081	4057
1956	20400	18000	16500	15000	13325	12140	10500	8540
1957	12800	11500	10255	9850	9233	8410	7193	6646
1958	11800	10000	8240	7587	7128	6726	6009	5161
1959	17600	16300	13650	11600	10805	9964	8800	6365
1960	14100	13000	10655	10203	9653	8922	7559	4869
1961	20400	15000	14000	12667	11625	11600	11571	9067
1962	11300	10100	8980	8037	7620	7668	7103	5469
1963	24800	20000	15250	12213	10203	8882	9739	7465
1964	22000	17300	14600	12497	11023	10046	8707	6627
1965	26600	22200	18650	17100	15675	14448	12266	7670
1966	10500	8970	8015	6947	6545	6110	5594	4791
1967	19100	16200	14300	11990	10685	9604	8106	6704
1968	12300	10600	9335	8687	8270	8130	7464	5127
1969	15200	12800	11750	11010	10250	9402	8109	5533
1970	13500	12800	12050	10930	10260	10138	9696	9003
1971	17800	15100	13200	11783	10265	9310	9270	8134
1972	38000	26800	26100	21900	18773	16776	13686	9407
1973	20600	15100	14350	13267	12850	11900	11421	8319
1974	33000	26100	24300	21967	19225	17342	14313	9740
1975	21200	20000	16750	15133	13725	12758	11639	8230
1976	32700	25900	23550	22467	20700	19260	16641	10811
1977	5420	4770	4140	3927	3648	3512	3237	2780
1978	43200	29400	25150	20800	17598	15818	13443	12347
1979	16400	11500	8675	7607	7068	6744	6684	5025
1980		8720	8590	8447	8093	7910	7467	4933
1981								
1982								
1983								
1984								
1985								
1986	29625	17393	15768	13793	12118	10705	8665	5838
1987	24942	19208	15099	13379	12395	11431	10100	6677
1988	21024	14144	10679	9353	8245	7536	6883	4890
1989	6431	5973	5796	5673	5490	5376	5171	4225
1990	21693	15864	15243	13666	12015	10978	9528	5856
1991	23796	19877	16724	14558	13226	12102	10500	6744
1992	10225	8668	8649	8486	8386	8092	7192	4916
1993	8878	7769	6713	5931	5340	4857	4217	3160
1994	7703	7091	6718	6345	6033	5843	5443	4492
1995	18348	13379	11802	10222	9308	8524	7293	6966
1996	59059	46158	37748	33097	28287	24789	19707	11517
1997	30103	23987	21311	20929	20307	19667	17140	11533
1998	20546	11372	10703	9668	8618	7741	7013	5512
1999	26567	20642	18014	15258	13298	11957	10883	8592
2000	22840	16915	16533	13749	11752	10277	8523	6637
2001	4453	3278	2929	2810	2640	2471	2193	1641
2002	22267	15768	13045	11337	10335	9920	8680	5939
2003	30772	23031	20069	16756	14468	12718	10384	6760
2004	16246	13283	11946	10776	9588	8605	7165	5047
2005	11659	9748	8367	7253	6465	5837	4919	3482
2006	21215	17966	17249	15545	14693	13742	11823	8983
2007	35550	28574	23127	20323	17436	15353	12855	8519
2008	22458	16342	15625	13258	11410	10017	8062	4848
2009	34117	27140	24512	20992	17909	15787	13069	8624

Notes:
Calculated using the drainage area ratio from the Toutle River at Tower Road gage

Toutle River near Silver Lake



Notes:
Period of Record for Regulated Flows Water Years 1927-2009.
Data from WYs 1981-1985 are not included in the analysis, these five years are part of the historical period.
Historic period for peak and 1-day curves is 1896-2009.
Historic period for 2- through 15-day curves is from 1927-2009.
Data are plotted using median plotting positins

COWLITZ RIVER STUDY
COLUMBIA RIVER BASIN

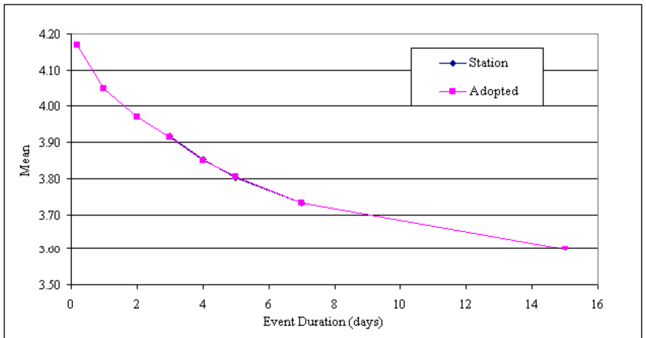
VOLUME DURATION FREQUENCY CURVES

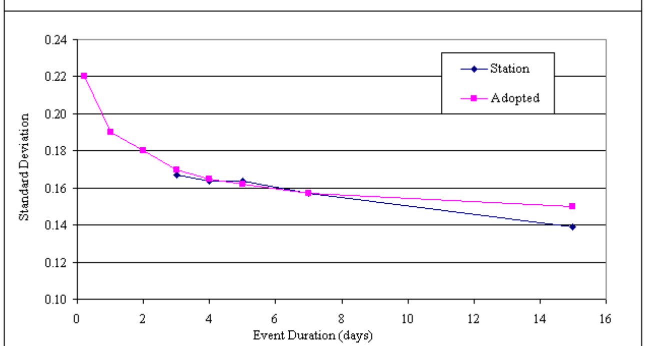
Toutle River near Silver Lake

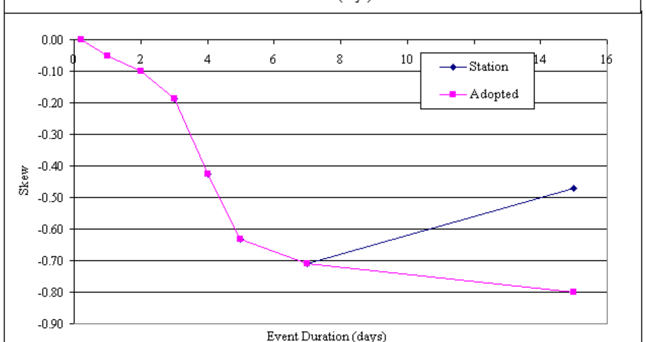
U.S ARMY CORPS OF ENGINEERS
Portland District, EC-HY

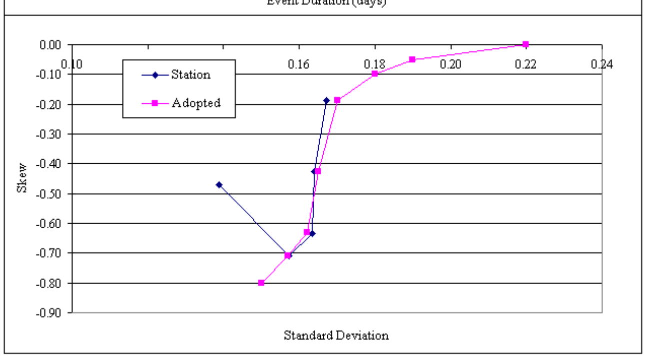
Castle Rock Locals

	0.20	1	2	3	4	5	7	15
Mean	#N/A	#N/A	#N/A	3.918	3.851	3.804	3.732	3.604
Standard Deviation	#N/A	#N/A	#N/A	0.167	0.164	0.164	0.157	0.139
Station Skew	#N/A	#N/A	#N/A	-0.188	-0.426	-0.632	-0.710	-0.471
N	0	0	0	71	71	71	71	67
H or "n"	114	0	0	73	73	73	73	73
L	0	0	0	2	2	2	2	6
Z	0	0	0	0	0	0	0	0
Low-outlier threshold								1,600
Adopted Mean	4.1700	4.0500	3.9700	3.915	3.851	3.804	3.732	3.604
Adopted Standard Deviation	0.2200	0.1900	0.1800	0.170	0.165	0.162	0.157	0.150
Adopted Skew	0.000	-0.050	-0.100	-0.188	-0.426	-0.632	-0.710	-0.800





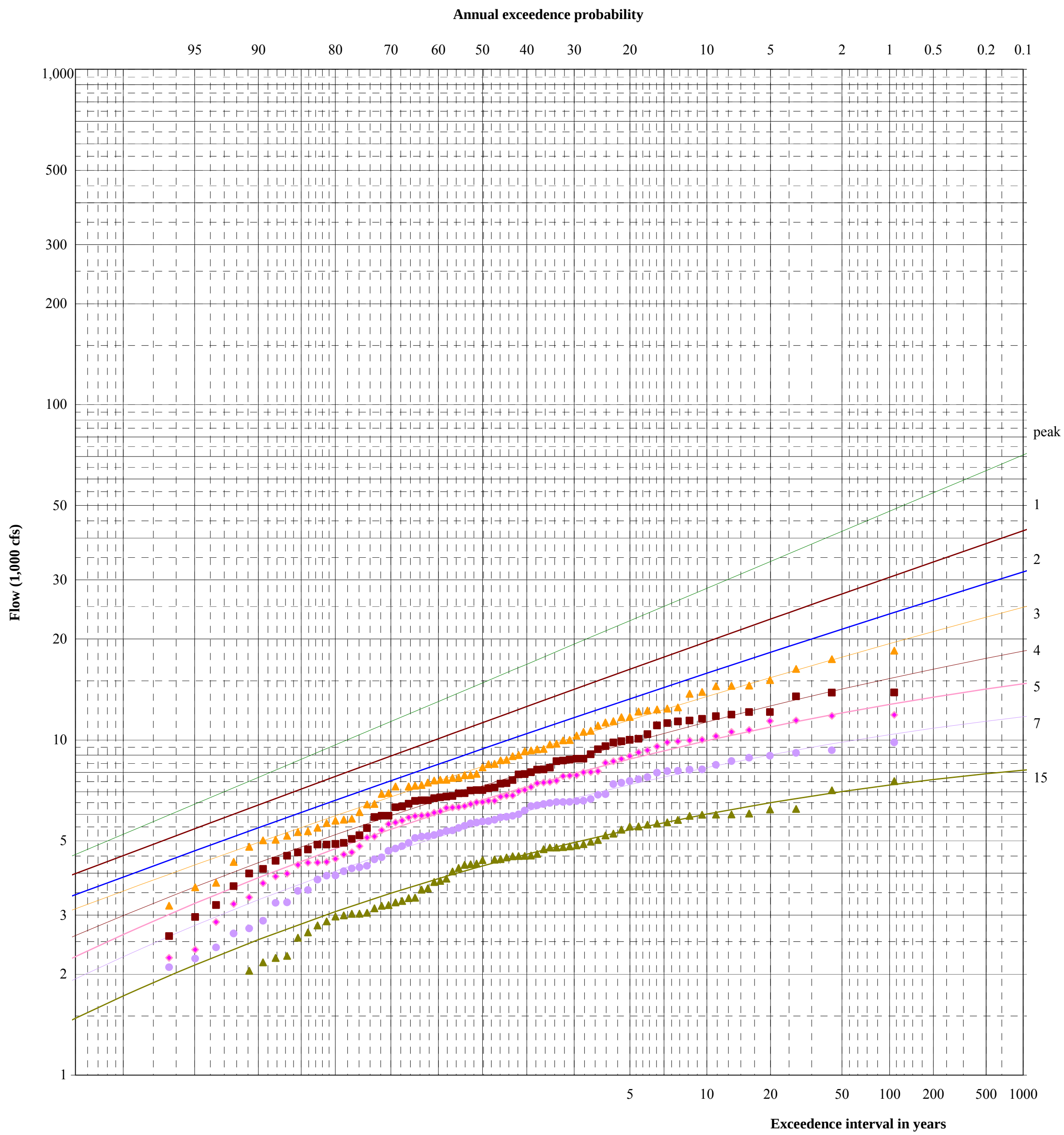




	Peak	1	2	3	4	5	7	15
1935				10289	8631	7424	6340	4378
1936				11022	11222	9552	7536	5983
1937				7260	6599	5943	4819	3767
1938				11697	10395	9169	8080	5520
1939				10556	9999	9313	7630	5022
1940				6897	5883	5605	5096	4237
1941				3201	2596	2237	2226	1530
1942				7333	6297	5669	4740	3381
1943				7704	6930	6431	5716	4050
1944				5034	3993	3391	2646	1539
1945				7854	6796	5925	4929	3273
1946				8690	8006	7237	5999	4719
1947				7293	6920	6520	5931	4501
1948				6941	6691	6299	5367	4770
1949				12133	10072	8769	7746	6222
1950				9742	8786	7844	6481	4788
1951				9687	7886	7023	5891	4389
1952				7256	6770	6264	5271	3854
1953				7711	7082	6823	6433	5931
1954				8279	7166	6767	5774	4389
1955				5746	4890	4326	3564	2569
1956				7260	6326	6118	5343	4257
1957				5471	4930	4548	3933	3028
1958				5306	4712	4307	3827	3143
1959				5647	5061	4816	4166	2993
1960				5170	4358	3914	3276	2170
1961				11367	9371	10259	9143	5582
1962				5335	4510	3988	4053	3307
1963				7605	6447	5743	4664	3573
1964				14450	11553	9841	8160	7070
1965				8459	7589	6578	5634	4148
1966				7843	7053	6580	5703	5186
1967				8719	7211	6281	5144	4885
1968				7458	6570	5972	5460	3605
1969				8936	7403	6819	6376	4811
1970				10644	9579	8543	7981	7530
1971				11271	9054	7814	6564	5977
1972				14520	12104	10562	8417	5512
1973				7916	6736	5871	5867	4455
1974				13897	11419	9962	8069	4505
1975				15070	12101	9902	7379	4967
1976				12320	11894	11868	9830	5767
1977				3630	2961	2367	2099	1453
1978				12393	11763	10693	8836	6201
1979				6435	5190	4616	4209	2873
1980				7557	6602	6058	5161	3216
1981								
1982				9375	8782	8059	7443	6027
1983				9984	7903	7459	6531	3794
1984				9244	8260	7554	5544	4411
1985				6090	5454	5111	4463	3030
1986				9964	8145	7086	5660	4276
1987				13722	11342	10012	8142	4841
1988				9295	7421	6339	5200	3371
1989				4326	3660	3241	2738	2236
1990				17399	13839	11785	9314	5394
1991				12495	9814	7995	6520	4568
1992				6421	5940	5378	4399	2795
1993				1813	1660	1611	1528	1217
1994				4799	4612	4414	3936	3050
1995				5011	4119	3735	2885	2047
1996				16283	13480	11372	8962	5625
1997				9018	8166	7491	6855	5251
1998								
1999				8479	7078	6505	6151	5673
2000				5782	4870	4301	3543	2664
2001				2032	1772	1595	1350	1044
2002				9395	8730	8014	6660	4498
2003				7602	5939	5152	4134	2971
2004				5819	4869	4239	3264	2268
2005				3746	3216	2861	2404	1518
2006				12234	9904	8629	6887	4770
2007				11673	8666	7792	6580	4517
2008				14493	11040	8943	6525	3198
2009				18452	13825	11426	8645	5991

Notes:
Data is model derived using daily data and no routing

Castle Rock Locals



Notes:
Period of Record for Regulated Flows Water Years 1935-2009, excluding 1981 & 1998
Data are plotted using median plotting positins

COWLITZ RIVER STUDY
COLUMBIA RIVER BASIN

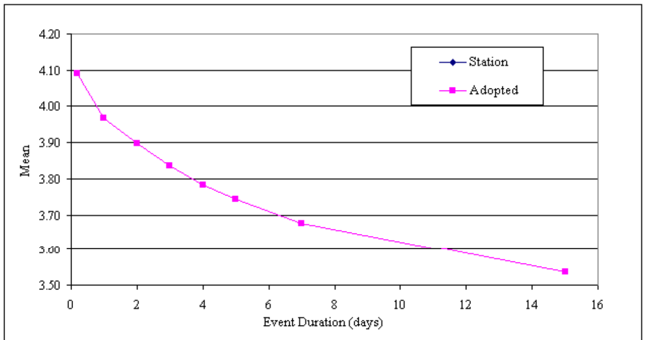
VOLUME DURATION FREQUENCY CURVES

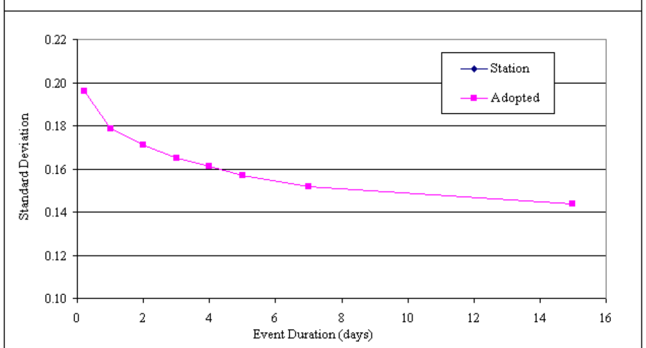
Castle Rock Locals

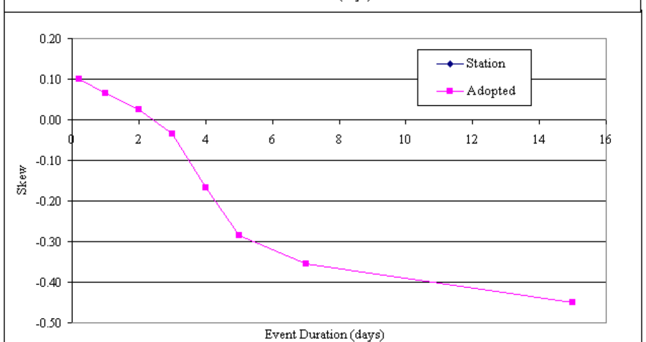
U.S ARMY CORPS OF ENGINEERS
Portland District, EC-HY

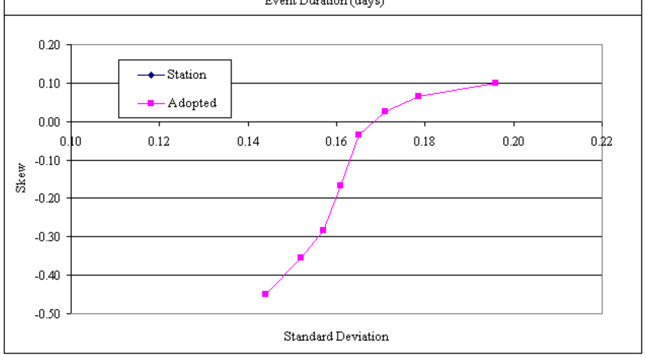
Reservoir Locals

	0.20	1	2	3	4	5	7	15	Peak	1	2	3	4	5	7	15
Mean	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A								
Standard Deviation	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A								
Station Skew	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A								
N	0	0	0	0	0	0	0	0								
H or "n"	0	0	0	0	0	0	0	0								
L	0	0	0	0	0	0	0	0								
Z	0	0	0	0	0	0	0	0								
Low-outlier threshold																
Adopted Mean	4.091	3.968	3.898	3.836	3.784	3.743	3.677	3.537								
Adopted Standard Deviation	0.196	0.179	0.171	0.165	0.161	0.157	0.152	0.144								
Adopted Skew	0.100	0.065	0.025	-0.034	-0.168	-0.285	-0.355	-0.450								



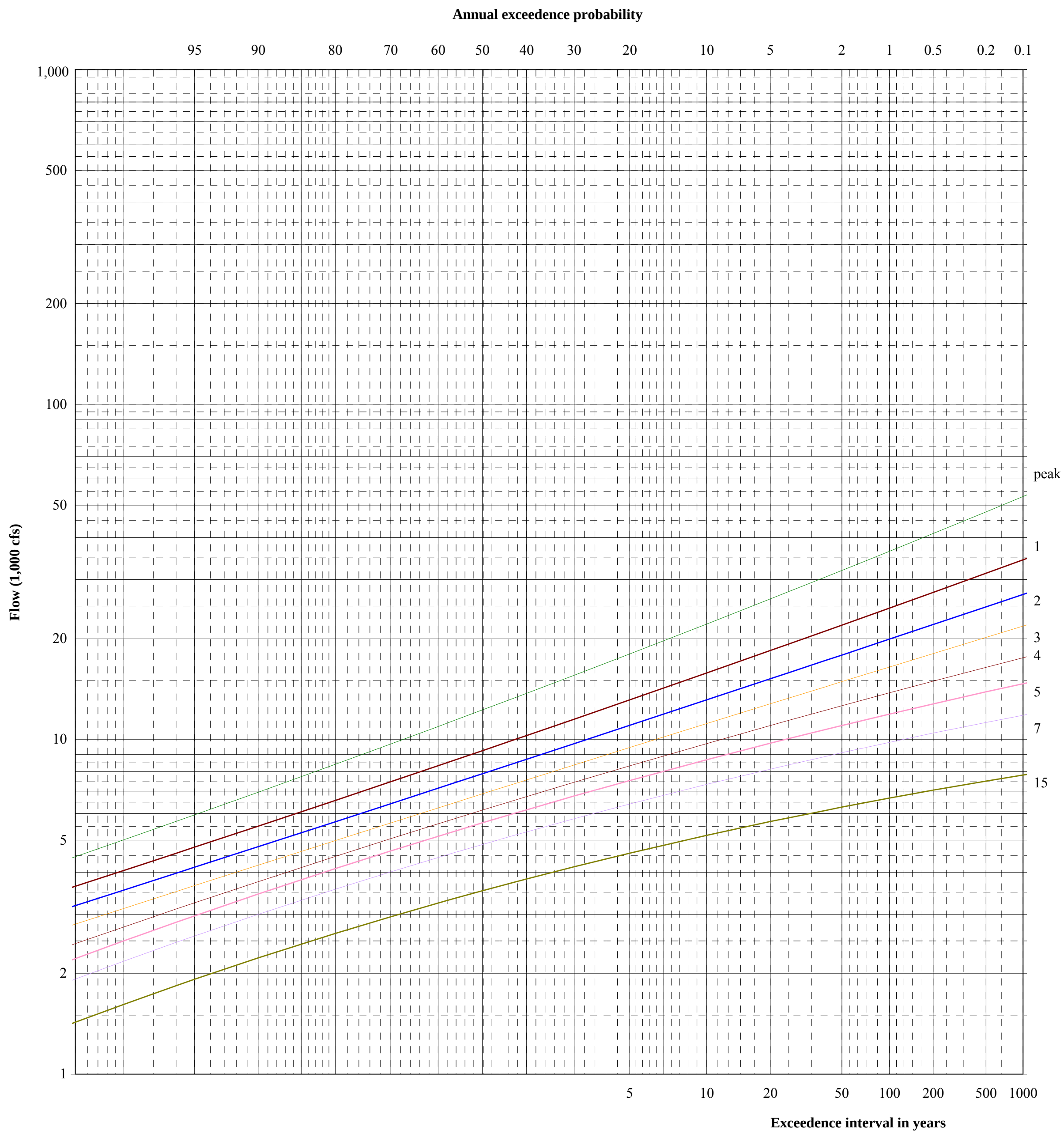






Notes:
Mean values are based on drainage area and average annual precipitation, correlated with other tributaries in this study.
Standard deviation and skew statistics are the average of CASLOC and TILT statistics.

Reservoir Locals



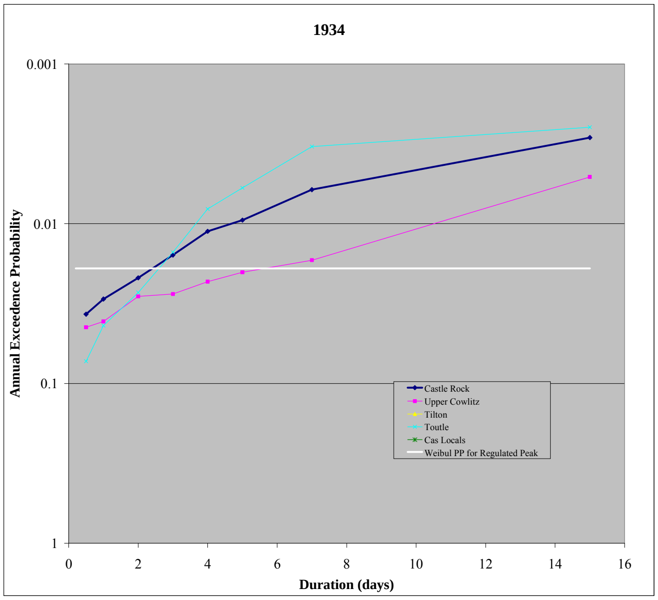
Notes:
Curves are created from synthetic statistics

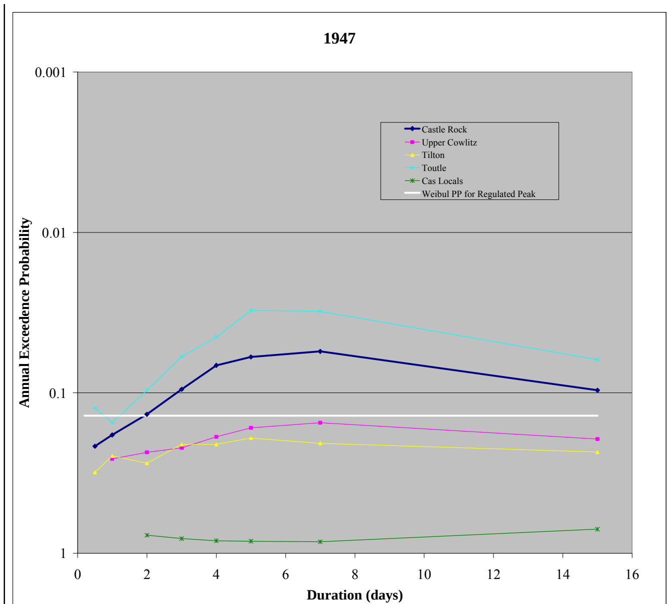
COWLITZ RIVER STUDY
COLUMBIA RIVER BASIN

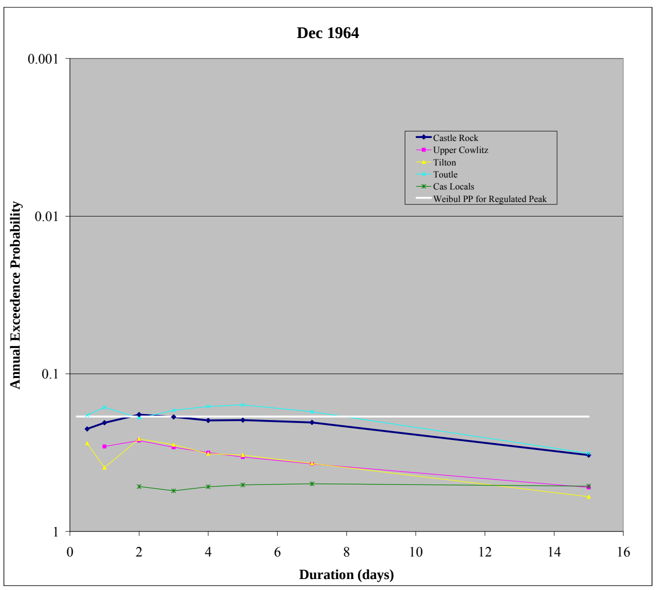
VOLUME DURATION FREQUENCY CURVES

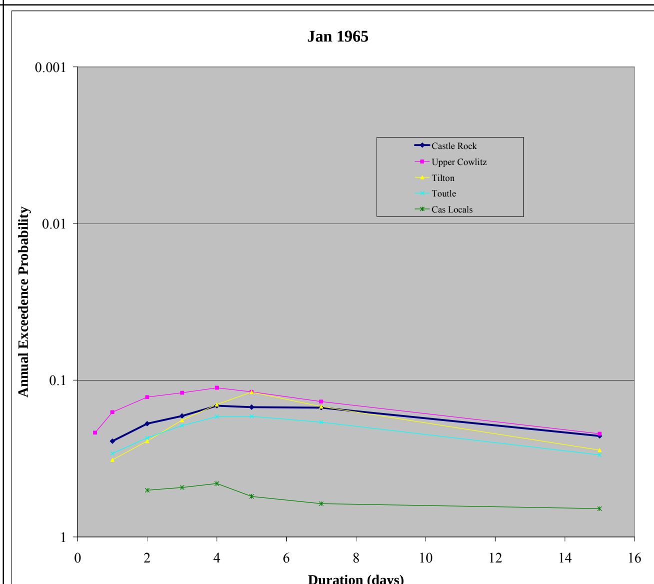
Reservoir Locals

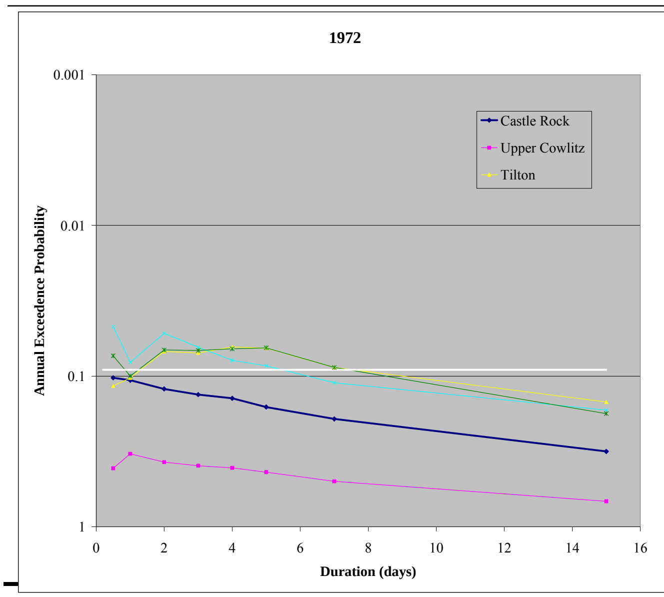
U.S ARMY CORPS OF ENGINEERS
Portland District, EC-HY

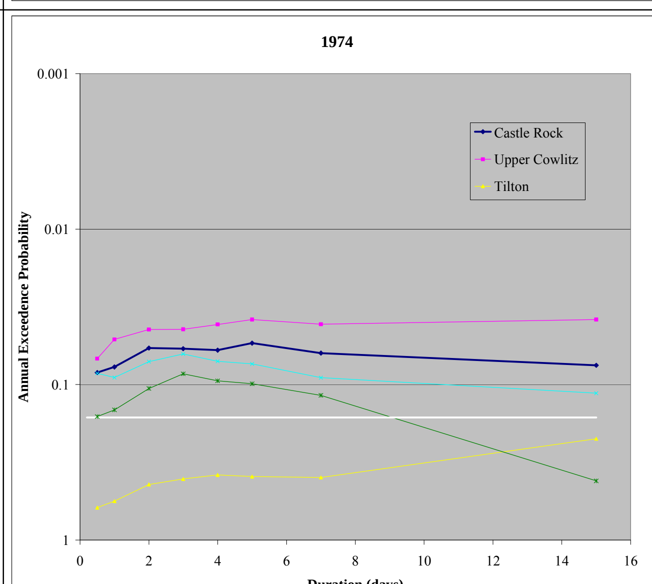

1934
Annual Exceedence Probability
0.001
0.01
0.1
1
0 2 4 6 8 10 12 14 16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals
Weibul PP for Regulated Peak

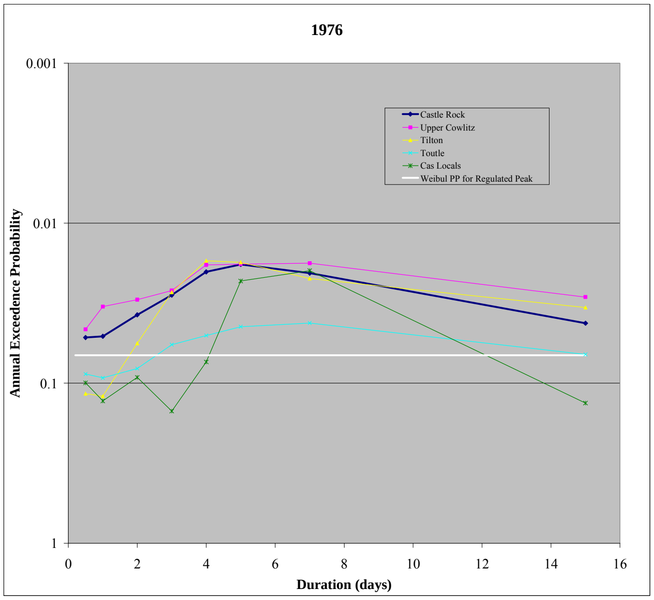

1947
Annual Exceedence Probability
0.001
0.01
0.1
1
0 2 4 6 8 10 12 14 16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals
Weibul PP for Regulated Peak


Dec 1964
Annual Exceedence Probability
0.001
0.01
0.1
1
0 2 4 6 8 10 12 14 16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals
Weibul PP for Regulated Peak

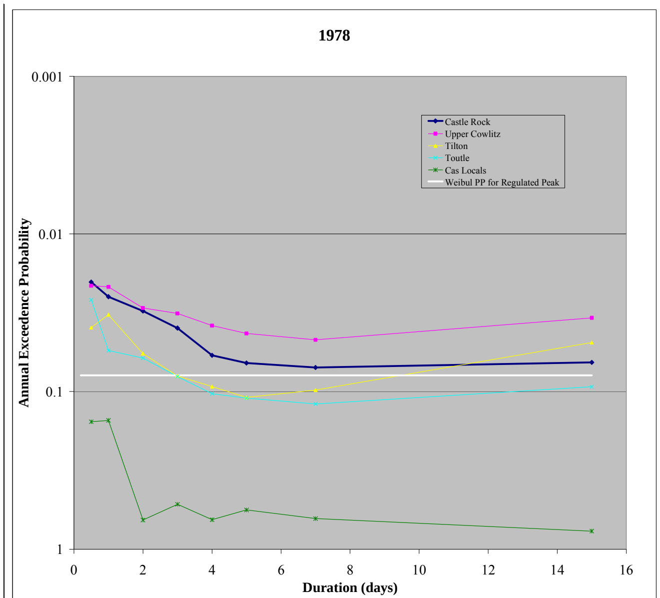

Jan 1965
Annual Exceedence Probability
0.001
0.01
0.1
1
0 2 4 6 8 10 12 14 16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals


1972
Annual Exceedence Probability
0.001
0.01
0.1
1
0 2 4 6 8 10 12 14 16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton

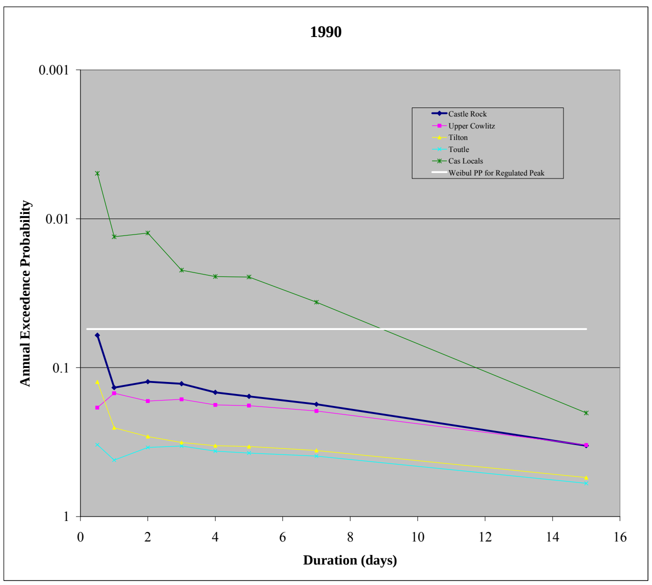

1974
Annual Exceedence Probability
0.001
0.01
0.1
1
0 2 4 6 8 10 12 14 16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton



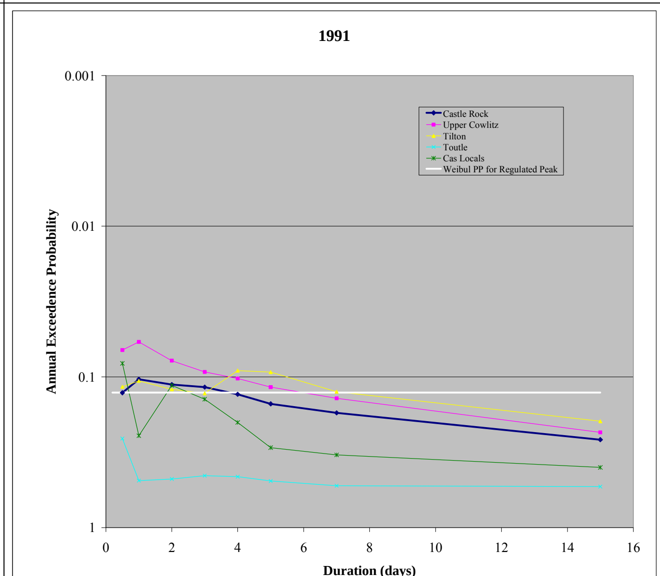
1976
Annual Exceedence Probability
0.001
0.01
0.1
1
0
2
4
6
8
10
12
14
16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals
Weibul PP for Regulated Peak



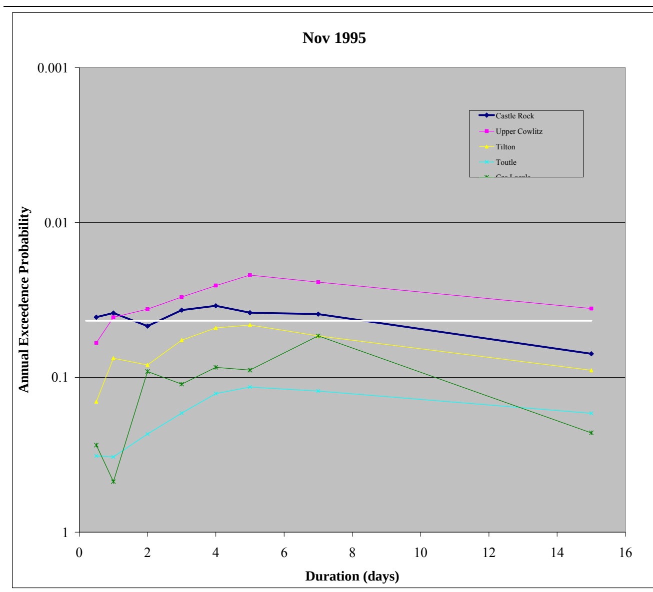
1978
Annual Exceedence Probability
0.001
0.01
0.1
1
0
2
4
6
8
10
12
14
16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals
Weibul PP for Regulated Peak



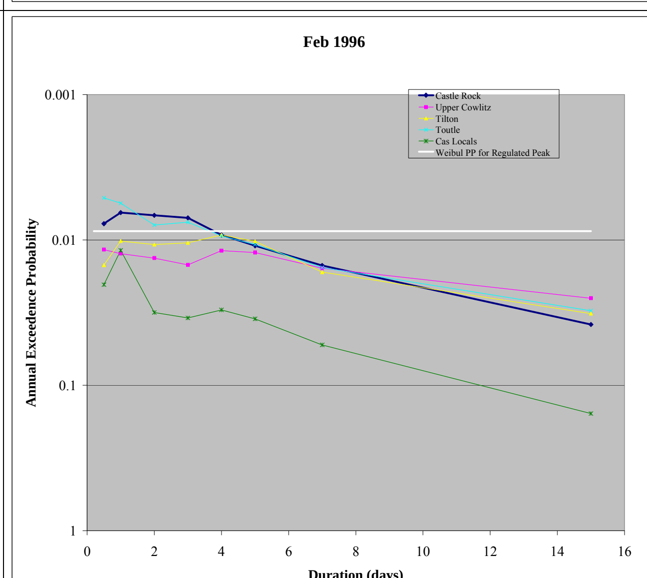
1990
Annual Exceedence Probability
0.001
0.01
0.1
1
0
2
4
6
8
10
12
14
16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals
Weibul PP for Regulated Peak



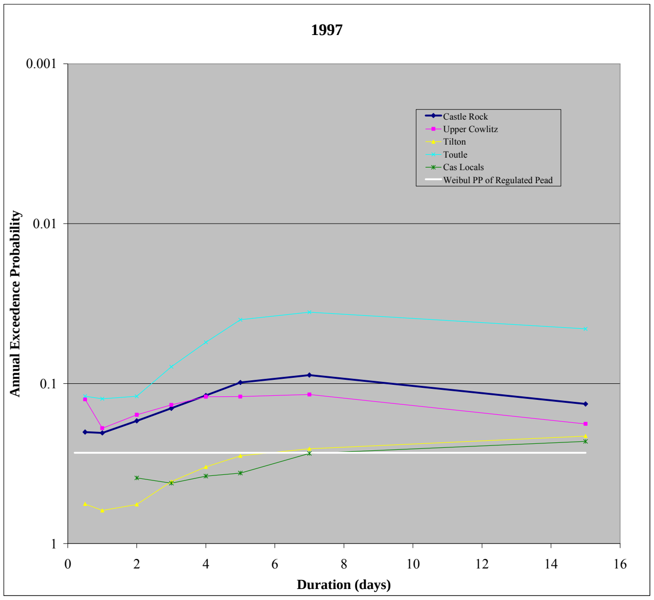
1991
Annual Exceedence Probability
0.001
0.01
0.1
1
0
2
4
6
8
10
12
14
16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals
Weibul PP for Regulated Peak

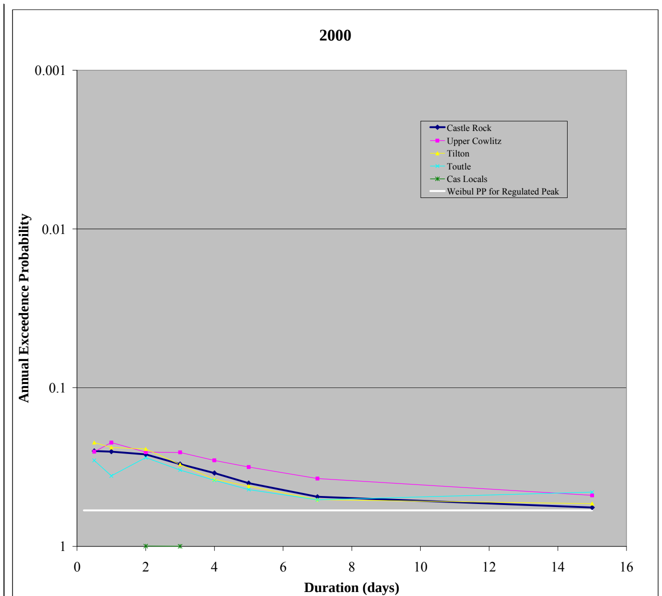


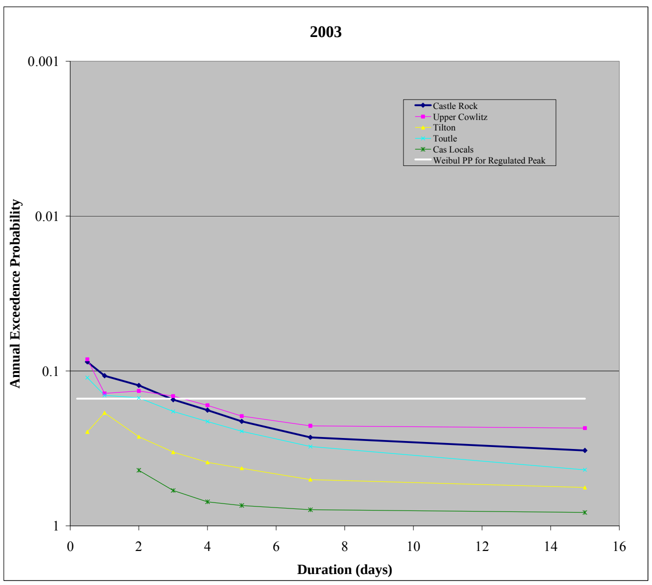
Nov 1995
Annual Exceedence Probability
0.001
0.01
0.1
1
0
2
4
6
8
10
12
14
16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle

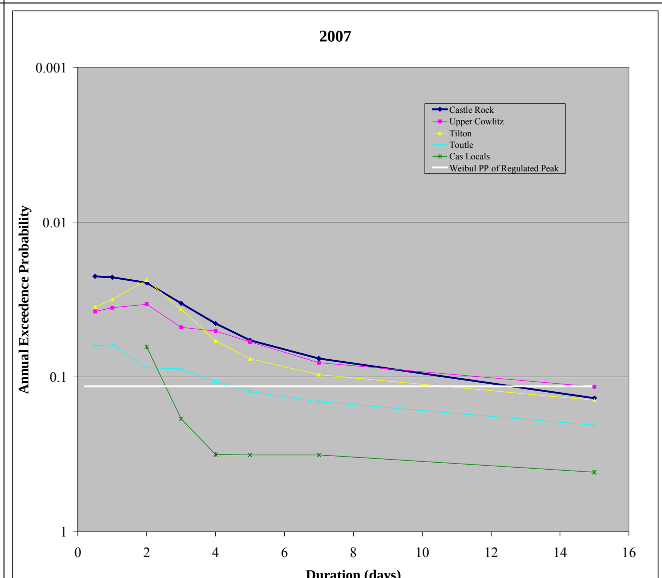


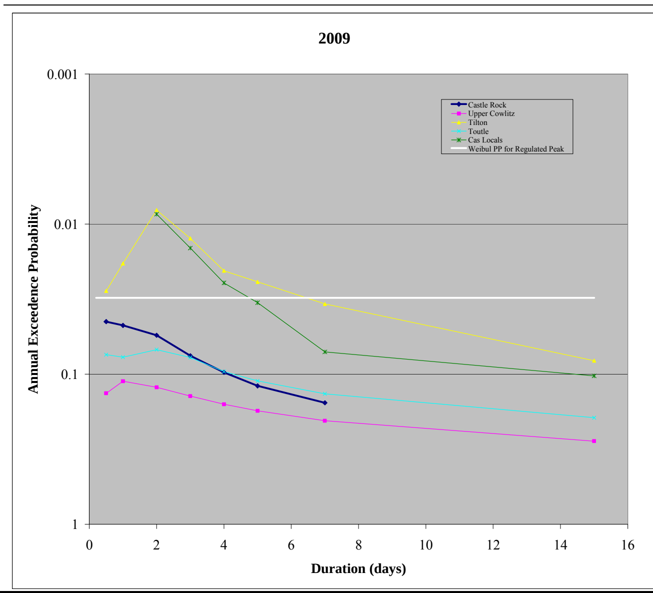
Feb 1996
Annual Exceedence Probability
0.001
0.01
0.1
1
0
2
4
6
8
10
12
14
16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals
Weibul PP for Regulated Peak


1997
Annual Exceedence Probability
0.001
0.01
0.1
1
0 2 4 6 8 10 12 14 16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals
Weibul PP of Regulated Pead


2000
Annual Exceedence Probability
0.001
0.01
0.1
1
0 2 4 6 8 10 12 14 16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals
Weibul PP for Regulated Peak


2003
Annual Exceedence Probability
0.001
0.01
0.1
1
0 2 4 6 8 10 12 14 16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals
Weibul PP for Regulated Peak


2007
Annual Exceedence Probability
0.001
0.01
0.1
1
0 2 4 6 8 10 12 14 16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals
Weibul PP of Regulated Peak


2009
Annual Exceedence Probability
0.001
0.01
0.1
1
0 2 4 6 8 10 12 14 16
Duration (days)
Castle Rock
Upper Cowlitz
Tilton
Toutle
Cas Locals
Weibul PP for Regulated Peak

Design AEP event 0.1
spatial pattern = 1 Upper Cowlitz
temporal pattern = 1 short
Min 1-3 AEP 0.0819

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0800	0.0640	0.1429	0.1488	0.4048	0.1881
1	0.0819	0.0640	0.1161	0.2381	0.4190	0.1964
2	0.0999	0.0840	0.1786	0.3125	0.4976	0.2381
3	0.1209	0.0920	0.2411	0.3423	0.5119	0.2798
4	0.1399	0.1040	0.2768	0.3869	0.5476	0.3298
5	0.1671	0.1160	0.3036	0.4464	0.5905	0.3714
7	0.2132	0.1400	0.3571	0.5357	0.6476	0.4548

Design AEP event 0.1
spatial pattern = 3 Highlands
temporal pattern = 1 short
Min 1-3 AEP 0.0883

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0921	0.0711	0.4700	0.0500	0.5429	0.3760
1	0.0883	0.0711	0.4475	0.0800	0.5486	0.3800
2	0.1012	0.0933	0.5000	0.1050	0.5800	0.4000
3	0.1161	0.1022	0.5525	0.1150	0.5857	0.4200
4	0.1298	0.1156	0.5825	0.1300	0.6000	0.4440
5	0.1505	0.1289	0.6050	0.1500	0.6171	0.4640
7	0.1843	0.1556	0.6500	0.1800	0.6400	0.5040

Design AEP event 0.1
spatial pattern = 1 Upper Cowlitz
temporal pattern = 2 medium
Min 1-3 AEP 0.0980

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.1106	0.0800	0.1676	0.2500	0.4706	0.2447
1	0.1023	0.0760	0.1941	0.3235	0.5082	0.2259
2	0.0980	0.0800	0.1500	0.2794	0.4329	0.2353
3	0.1028	0.0840	0.1853	0.2794	0.4706	0.2447
4	0.1072	0.0864	0.1976	0.2853	0.5082	0.2541
5	0.1181	0.0880	0.2082	0.2941	0.5459	0.2729
7	0.1366	0.0960	0.2294	0.3235	0.6212	0.2918

Design AEP event 0.1
spatial pattern = 3 Highlands
temporal pattern = 2 medium
Min 1-3 AEP 0.0965

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.1082	0.0941	0.2388	0.0900	0.6667	0.3367
1	0.1017	0.0894	0.2725	0.1165	0.6800	0.3300
2	0.0965	0.0941	0.2163	0.1006	0.6533	0.3333
3	0.1005	0.0988	0.2613	0.1006	0.6667	0.3367
4	0.1031	0.1016	0.2770	0.1027	0.6800	0.3400
5	0.1115	0.1035	0.2905	0.1059	0.6933	0.3467
7	0.1257	0.1129	0.3175	0.1165	0.7200	0.3533

Design AEP event 0.1
spatial pattern = 1 Upper Cowlitz
temporal pattern = 3 long
Min 1-3 AEP 0.0815

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.1350	0.1200	0.2898	0.3125	0.5455	0.3182
1	0.1222	0.0920	0.2216	0.3338	0.5455	0.2727
2	0.0972	0.0760	0.1619	0.2841	0.4182	0.2273
3	0.0815	0.0680	0.1278	0.2344	0.4000	0.1818
4	0.0699	0.0600	0.1023	0.2060	0.4000	0.1364
5	0.0662	0.0520	0.0852	0.1918	0.4000	0.1136
7	0.0644	0.0480	0.0852	0.1918	0.4091	0.1136

Design AEP event 0.1
spatial pattern = 3 Highlands
temporal pattern = 3 long
Min 1-3 AEP 0.0793

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.1323	0.1446	0.5100	0.1258	0.6000	0.5280
1	0.1228	0.1108	0.3900	0.1388	0.6000	0.4640
2	0.0968	0.0916	0.2850	0.1084	0.4600	0.4000
3	0.0793	0.0819	0.2250	0.0781	0.4400	0.3360
4	0.0665	0.0723	0.1800	0.0607	0.4400	0.2720
5	0.0617	0.0627	0.1500	0.0520	0.4400	0.2400
7	0.0597	0.0578	0.1500	0.0520	0.4500	0.2400

Design AEP event 0.1
spatial pattern = 2 Toutle
temporal pattern = 1 short
Min 1-3 AEP 0.0836

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0762	0.1304	0.1739	0.0400	0.2609	0.2473
1	0.0836	0.1304	0.1413	0.0640	0.2739	0.2514
2	0.1008	0.1712	0.2174	0.0840	0.3457	0.2717
3	0.1206	0.1875	0.2935	0.0920	0.3587	0.2921
4	0.1376	0.2120	0.3370	0.1040	0.3913	0.3166
5	0.1617	0.2364	0.3696	0.1200	0.4304	0.3370
7	0.2018	0.2853	0.4348	0.1440	0.4826	0.3777

Design AEP event 0.1
spatial pattern = 4 Lowlands
temporal pattern = 1 short
Min 1-3 AEP 0.0760

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0559	0.2791	0.0480	0.0872	0.0250	0.0659
1	0.0760	0.2791	0.0390	0.1395	0.0300	0.0706
2	0.1027	0.3663	0.0600	0.1831	0.0575	0.0941
3	0.1309	0.4012	0.0810	0.2006	0.0625	0.1176
4	0.1553	0.4535	0.0930	0.2267	0.0750	0.1459
5	0.1875	0.5058	0.1020	0.2616	0.0900	0.1694
7	0.2436	0.6105	0.1200	0.3140	0.1100	0.2165

Design AEP event 0.1
spatial pattern = 2 Toutle
temporal pattern = 2 medium
Min 1-3 AEP 0.0974

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.1035	0.1579	0.2021	0.0680	0.3158	0.2737
1	0.1027	0.1500	0.2274	0.0880	0.3442	0.2526
2	0.0974	0.1579	0.1853	0.0760	0.2874	0.2632
3	0.1025	0.1658	0.2189	0.0760	0.3158	0.2737
4	0.1065	0.1705	0.2307	0.0776	0.3442	0.2842
5	0.1162	0.1737	0.2408	0.0800	0.3726	0.3053
7	0.1331	0.1895	0.2611	0.0880	0.4295	0.3263

Design AEP event 0.1
spatial pattern = uniform
temporal pattern = flat
Min 1-3 AEP 0.0975

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0990	0.1639	0.1639	0.1639	0.1639	0.1639
1	0.1021	0.1639	0.1639	0.1639	0.1639	0.1639
2	0.0975	0.1639	0.1639	0.1639	0.1639	0.1639
3	0.1005	0.1639	0.1639	0.1639	0.1639	0.1639
4	0.0996	0.1639	0.1639	0.1639	0.1639	0.1639
5	0.1042	0.1639	0.1639	0.1639	0.1639	0.1639
7	0.1094	0.1639	0.1639	0.1639	0.1639	0.1639

Design AEP event 0.1
spatial pattern = 2 Toutle
temporal pattern = 3 long
Min 1-3 AEP 0.0791

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.1279	0.2446	0.3696	0.0960	0.4402	0.3804
1	0.1239	0.1875	0.2826	0.1080	0.4402	0.3261
2	0.0998	0.1549	0.2065	0.0800	0.2804	0.2717
3	0.0791	0.1386	0.1630	0.0520	0.2576	0.2174
4	0.0642	0.1223	0.1304	0.0360	0.2576	0.1630
5	0.0578	0.1060	0.1087	0.0280	0.2576	0.1359
7	0.0559	0.0978	0.1087	0.0280	0.2690	0.1359

Design AEP event 0.02
spatial pattern = 1 Upper Cowlitz
temporal pattern = 1 short
Min 1-3 AEP 0.0158

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0164	0.0128	0.0453	0.0472	0.0981	0.0528
1	0.0158	0.0128	0.0368	0.0755	0.1087	0.0566
2	0.0196	0.0168	0.0566	0.0991	0.1668	0.0755
3	0.0243	0.0184	0.0764	0.1085	0.1774	0.0943
4	0.0285	0.0208	0.0877	0.1226	0.2038	0.1170
5	0.0350	0.0232	0.0962	0.1415	0.2355	0.1358
7	0.0449	0.0280	0.1132	0.1698	0.2777	0.1736

Design AEP event 0.02
spatial pattern = 3 Highlands
temporal pattern = 1 short
Min 1-3 AEP 0.0161

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0169	0.0160	0.0800	0.0113	0.2000	0.1053
1	0.0161	0.0160	0.0650	0.0180	0.2133	0.1100
2	0.0196	0.0210	0.1000	0.0236	0.2867	0.1333
3	0.0236	0.0230	0.1350	0.0259	0.3000	0.1567
4	0.0273	0.0260	0.1550	0.0293	0.3333	0.1847
5	0.0330	0.0290	0.1700	0.0338	0.3733	0.2080
7	0.0419	0.0350	0.2000	0.0405	0.4267	0.2547

Design AEP event 0.02
spatial pattern = 1 Upper Cowlitz
temporal pattern = 2 medium
Min 1-3 AEP 0.0187

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0232	0.0133	0.0582	0.0867	0.1633	0.0857
1	0.0203	0.0127	0.0673	0.1122	0.1796	0.0776
2	0.0187	0.0133	0.0520	0.0969	0.1469	0.0816
3	0.0195	0.0140	0.0643	0.0969	0.1633	0.0857
4	0.0203	0.0144	0.0686	0.0990	0.1796	0.0898
5	0.0228	0.0147	0.0722	0.1020	0.1959	0.0980
7	0.0262	0.0160	0.0796	0.1122	0.2286	0.1061

Design AEP event 0.02
spatial pattern = 3 Highlands
temporal pattern = 2 medium
Min 1-3 AEP 0.0190

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0242	0.0168	0.1140	0.0161	0.4000	0.1640
1	0.0206	0.0160	0.1320	0.0208	0.4200	0.1560
2	0.0190	0.0168	0.1020	0.0180	0.3800	0.1600
3	0.0196	0.0177	0.1260	0.0180	0.4000	0.1640
4	0.0201	0.0182	0.1344	0.0184	0.4200	0.1680
5	0.0222	0.0185	0.1416	0.0189	0.4400	0.1760
7	0.0251	0.0202	0.1560	0.0208	0.4800	0.1840

Design AEP event 0.02
spatial pattern = 1 Upper Cowlitz
temporal pattern = 3 long
Min 1-3 AEP 0.0146

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0325	0.0240	0.0810	0.0952	0.1905	0.0889
1	0.0260	0.0184	0.0619	0.1071	0.1905	0.0762
2	0.0186	0.0152	0.0452	0.0794	0.1016	0.0635
3	0.0146	0.0136	0.0357	0.0516	0.0889	0.0508
4	0.0117	0.0120	0.0286	0.0357	0.0889	0.0381
5	0.0107	0.0104	0.0238	0.0278	0.0889	0.0317
7	0.0102	0.0096	0.0238	0.0278	0.0952	0.0317

Design AEP event 0.02
spatial pattern = 3 Highlands
temporal pattern = 3 long
Min 1-3 AEP 0.0142

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0307	0.0253	0.2040	0.0227	0.4000	0.2240
1	0.0255	0.0194	0.1560	0.0256	0.4000	0.1920
2	0.0181	0.0160	0.1140	0.0189	0.2880	0.1600
3	0.0142	0.0143	0.0900	0.0123	0.2720	0.1280
4	0.0115	0.0126	0.0720	0.0085	0.2720	0.0960
5	0.0104	0.0109	0.0600	0.0066	0.2720	0.0800
7	0.0098	0.0101	0.0600	0.0066	0.2800	0.0800

Design AEP event 0.02
spatial pattern = 2 Toutle
temporal pattern = 1 short
Min 1-3 AEP 0.0157

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0138	0.0333	0.0444	0.0080	0.0417	0.0486
1	0.0157	0.0333	0.0361	0.0128	0.0500	0.0521
2	0.0199	0.0438	0.0556	0.0168	0.0958	0.0694
3	0.0246	0.0479	0.0750	0.0184	0.1042	0.0868
4	0.0287	0.0542	0.0861	0.0208	0.1250	0.1076
5	0.0348	0.0604	0.0944	0.0240	0.1500	0.1250
7	0.0440	0.0729	0.1111	0.0288	0.1833	0.1597

Design AEP event 0.02
spatial pattern = 4 Lowlands
temporal pattern = 1 short
Min 1-3 AEP 0.0147

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0113	0.0658	0.0096	0.0205	0.0060	0.0153
1	0.0147	0.0658	0.0078	0.0329	0.0068	0.0164
2	0.0198	0.0863	0.0120	0.0432	0.0112	0.0219
3	0.0251	0.0945	0.0162	0.0473	0.0120	0.0274
4	0.0293	0.1068	0.0186	0.0534	0.0140	0.0340
5	0.0354	0.1192	0.0204	0.0616	0.0164	0.0395
7	0.0445	0.1438	0.0240	0.0740	0.0196	0.0504

Design AEP event 0.02
spatial pattern = 2 Toutle
temporal pattern = 2 medium
Min 1-3 AEP 0.0186

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0213	0.0400	0.0507	0.0108	0.0800	0.0700
1	0.0205	0.0380	0.0587	0.0160	0.0880	0.0633
2	0.0186	0.0400	0.0453	0.0129	0.0720	0.0667
3	0.0195	0.0420	0.0560	0.0129	0.0800	0.0700
4	0.0202	0.0432	0.0597	0.0133	0.0880	0.0733
5	0.0223	0.0440	0.0629	0.0139	0.0960	0.0800
7	0.0254	0.0480	0.0693	0.0160	0.1120	0.0867

Design AEP event 0.02
spatial pattern = uniform
temporal pattern = flat
Min 1-3 AEP 0.0189

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0210	0.0370	0.0370	0.0370	0.0370	0.0370
1	0.0203	0.0370	0.0370	0.0370	0.0370	0.0370
2	0.0189	0.0370	0.0370	0.0370	0.0370	0.0370
3	0.0194	0.0370	0.0370	0.0370	0.0370	0.0370
4	0.0190	0.0370	0.0370	0.0370	0.0370	0.0370
5	0.0201	0.0370	0.0370	0.0370	0.0370	0.0370
7	0.0207	0.0370	0.0370	0.0370	0.0370	0.0370

Design AEP event 0.02
spatial pattern = 2 Toutle
temporal pattern = 3 long
Min 1-3 AEP 0.0148

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0274	0.0563	0.0850	0.0192	0.1013	0.0875
1	0.0250	0.0431	0.0650	0.0216	0.1013	0.0750
2	0.0189	0.0356	0.0475	0.0160	0.0645	0.0625
3	0.0148	0.0319	0.0375	0.0104	0.0593	0.0500
4	0.0118	0.0281	0.0300	0.0072	0.0593	0.0375
5	0.0106	0.0244	0.0250	0.0056	0.0593	0.0313
7	0.0101	0.0225	0.0250	0.0056	0.0619	0.0313

Design AEP event	0.01	
spatial pattern =	1	Upper Cowlitz
temporal pattern =	1	short
Min 1-3 AEP	0.0077	

	Castle Rock	Jpper Cowlit	Tilton	Toutle	Cas Locals	eservoir Locals
0	0.0082	0.0064	0.0267	0.0278	0.0578	0.0311
1	0.0077	0.0064	0.0217	0.0444	0.0640	0.0333
2	0.0096	0.0084	0.0333	0.0583	0.0982	0.0444
3	0.0119	0.0092	0.0450	0.0639	0.1044	0.0556
4	0.0139	0.0104	0.0517	0.0722	0.1200	0.0689
5	0.0172	0.0116	0.0567	0.0833	0.1387	0.0800
7	0.0219	0.0140	0.0667	0.1000	0.1636	0.1022

Design AEP event	0.01	
spatial pattern =	3	Highlands
temporal pattern =	1	short
Min 1-3 AEP	0.0078	

	Castle Rock	Jpper Cowlit	Tilton	Toutle	Cas Locals	eservoir Locals
0	0.0079	0.0091	0.0400	0.0064	0.0667	0.0467
1	0.0078	0.0091	0.0325	0.0103	0.0800	0.0500
2	0.0097	0.0120	0.0500	0.0135	0.1533	0.0667
3	0.0119	0.0131	0.0675	0.0148	0.1667	0.0833
4	0.0139	0.0149	0.0775	0.0167	0.2000	0.1033
5	0.0171	0.0166	0.0850	0.0193	0.2400	0.1200
7	0.0218	0.0200	0.1000	0.0231	0.2933	0.1533

Design AEP event	0.01	
spatial pattern =	1	Upper Cowlitz
temporal pattern =	2	medium
Min 1-3 AEP	0.0091	

	Castle Rock	Jpper Cowlit	Tilton	Toutle	Cas Locals	eservoir Locals
0	0.0121	0.0075	0.0291	0.0434	0.0816	0.0429
1	0.0100	0.0071	0.0337	0.0561	0.0898	0.0388
2	0.0091	0.0075	0.0260	0.0485	0.0735	0.0408
3	0.0094	0.0079	0.0321	0.0485	0.0816	0.0429
4	0.0098	0.0081	0.0343	0.0495	0.0898	0.0449
5	0.0110	0.0082	0.0361	0.0510	0.0980	0.0490
7	0.0125	0.0090	0.0398	0.0561	0.1143	0.0531

Design AEP event	0.01	
spatial pattern =	3	Highlands
temporal pattern =	2	medium
Min 1-3 AEP	0.0093	

	Castle Rock	Jpper Cowlit	Tilton	Toutle	Cas Locals	eservoir Locals
0	0.0126	0.0094	0.0570	0.0090	0.2000	0.0840
1	0.0102	0.0089	0.0660	0.0116	0.2200	0.0760
2	0.0093	0.0094	0.0510	0.0101	0.1800	0.0800
3	0.0096	0.0099	0.0630	0.0101	0.2000	0.0840
4	0.0099	0.0102	0.0672	0.0103	0.2200	0.0880
5	0.0110	0.0104	0.0708	0.0106	0.2400	0.0960
7	0.0124	0.0113	0.0780	0.0116	0.2800	0.1040

Design AEP event	0.01	
spatial pattern =	1	Upper Cowlitz
temporal pattern =	3	long
Min 1-3 AEP	0.0066	

	Castle Rock	Jpper Cowlit	Tilton	Toutle	Cas Locals	eservoir Locals
0	0.0169	0.0120	0.0425	0.0508	0.1067	0.0493
1	0.0128	0.0092	0.0325	0.0577	0.1067	0.0413
2	0.0087	0.0076	0.0238	0.0417	0.0507	0.0333
3	0.0066	0.0068	0.0188	0.0256	0.0427	0.0253
4	0.0052	0.0060	0.0150	0.0165	0.0427	0.0173
5	0.0047	0.0052	0.0125	0.0119	0.0427	0.0133
7	0.0043	0.0048	0.0125	0.0119	0.0467	0.0133

Design AEP event	0.01	
spatial pattern =	3	Highlands
temporal pattern =	3	long
Min 1-3 AEP	0.0069	

	Castle Rock	Jpper Cowlit	Tilton	Toutle	Cas Locals	eservoir Locals
0	0.0164	0.0141	0.1020	0.0127	0.2400	0.1120
1	0.0129	0.0108	0.0780	0.0143	0.2400	0.0960
2	0.0090	0.0089	0.0570	0.0106	0.1280	0.0800
3	0.0069	0.0080	0.0450	0.0069	0.1120	0.0640
4	0.0055	0.0071	0.0360	0.0048	0.1120	0.0480
5	0.0050	0.0061	0.0300	0.0037	0.1120	0.0400
7	0.0047	0.0056	0.0300	0.0037	0.1200	0.0400

Design AEP event	0.01	
spatial pattern =	2	Toutle
temporal pattern =	1	short
Min 1-3 AEP	0.0076	

	Castle Rock	Jpper Cowlit	Tilton	Toutle	Cas Locals	eservoir Locals
0	0.0069	0.0185	0.0246	0.0040	0.0231	0.0269
1	0.0076	0.0185	0.0200	0.0064	0.0277	0.0288
2	0.0097	0.0242	0.0308	0.0084	0.0531	0.0385
3	0.0121	0.0265	0.0415	0.0092	0.0577	0.0481
4	0.0141	0.0300	0.0477	0.0104	0.0692	0.0596
5	0.0171	0.0335	0.0523	0.0120	0.0831	0.0692
7	0.0215	0.0404	0.0615	0.0144	0.1015	0.0885

Design AEP event	0.01	
spatial pattern =	4	Lowlands
temporal pattern =	1	short
Min 1-3 AEP	0.0070	

	Castle Rock	Jpper Cowlit	Tilton	Toutle	Cas Locals	eservoir Locals
0	0.0053	0.0343	0.0048	0.0107	0.0025	0.0080
1	0.0070	0.0343	0.0039	0.0171	0.0030	0.0086
2	0.0096	0.0450	0.0060	0.0225	0.0058	0.0114
3	0.0122	0.0493	0.0081	0.0246	0.0063	0.0143
4	0.0143	0.0557	0.0093	0.0279	0.0075	0.0177
5	0.0174	0.0621	0.0102	0.0321	0.0090	0.0206
7	0.0216	0.0750	0.0120	0.0386	0.0110	0.0263

Design AEP event	0.01	
spatial pattern =	2	Toutle
temporal pattern =	2	medium
Min 1-3 AEP	0.0091	

	Castle Rock	Jpper Cowlit	Tilton	Toutle	Cas Locals	eservoir Locals
0	0.0107	0.0231	0.0292	0.0041	0.0462	0.0404
1	0.0101	0.0219	0.0338	0.0061	0.0508	0.0365
2	0.0091	0.0231	0.0262	0.0049	0.0415	0.0385
3	0.0095	0.0242	0.0323	0.0049	0.0462	0.0404
4	0.0097	0.0249	0.0345	0.0051	0.0508	0.0423
5	0.0107	0.0254	0.0363	0.0053	0.0554	0.0462
7	0.0121	0.0277	0.0400	0.0061	0.0646	0.0500

Design AEP event	0.01	
spatial pattern =		uniform
temporal pattern =		flat
Min 1-3 AEP	0.0094	

	Castle Rock	Jpper Cowlit	Tilton	Toutle	Cas Locals	eservoir Locals
0	0.0108	0.0196	0.0196	0.0196	0.0196	0.0196
1	0.0102	0.0196	0.0196	0.0196	0.0196	0.0196
2	0.0094	0.0196	0.0196	0.0196	0.0196	0.0196
3	0.0096	0.0196	0.0196	0.0196	0.0196	0.0196
4	0.0094	0.0196	0.0196	0.0196	0.0196	0.0196
5	0.0099	0.0196	0.0196	0.0196	0.0196	0.0196
7	0.0101	0.0196	0.0196	0.0196	0.0196	0.0196

Design AEP event	0.01	
spatial pattern =	2	Toutle
temporal pattern =	3	long
Min 1-3 AEP	0.0073	

	Castle Rock	Jpper Cowlit	Tilton	Toutle	Cas Locals	eservoir Locals
0	0.0144	0.0300	0.0453	0.0096	0.0540	0.0467
1	0.0126	0.0230	0.0347	0.0108	0.0540	0.0400
2	0.0093	0.0190	0.0253	0.0080	0.0344	0.0333
3	0.0073	0.0170	0.0200	0.0052	0.0316	0.0267
4	0.0058	0.0150	0.0160	0.0036	0.0316	0.0200
5	0.0052	0.0130	0.0133	0.0028	0.0316	0.0167
7	0.0048	0.0120	0.0133	0.0028	0.0330	0.0167

Design AEP event	0.005	
spatial pattern =	1	Upper Cowlitz
temporal pattern =	1	short
Min 1-3 AEP	0.0038	

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0042	0.0032	0.0162	0.0169	0.0351	0.0189
1	0.0038	0.0032	0.0132	0.0270	0.0389	0.0203
2	0.0047	0.0042	0.0203	0.0355	0.0597	0.0270
3	0.0059	0.0046	0.0274	0.0389	0.0635	0.0338
4	0.0069	0.0052	0.0314	0.0439	0.0730	0.0419
5	0.0086	0.0058	0.0345	0.0507	0.0843	0.0486
7	0.0109	0.0070	0.0405	0.0608	0.0995	0.0622

Design AEP event	0.005	
spatial pattern =	3	Highlands
temporal pattern =	1	short
Min 1-3 AEP	0.0038	

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0041	0.0044	0.0300	0.0031	0.0500	0.0350
1	0.0038	0.0044	0.0244	0.0049	0.0600	0.0375
2	0.0047	0.0058	0.0375	0.0065	0.1150	0.0500
3	0.0058	0.0063	0.0506	0.0071	0.1250	0.0625
4	0.0068	0.0071	0.0581	0.0080	0.1500	0.0775
5	0.0084	0.0079	0.0638	0.0092	0.1800	0.0900
7	0.0106	0.0096	0.0750	0.0111	0.2200	0.1150

Design AEP event	0.005	
spatial pattern =	1	Upper Cowlitz
temporal pattern =	2	medium
Min 1-3 AEP	0.0043	

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0060	0.0040	0.0143	0.0213	0.0400	0.0210
1	0.0048	0.0038	0.0165	0.0275	0.0440	0.0190
2	0.0043	0.0040	0.0128	0.0238	0.0360	0.0200
3	0.0044	0.0042	0.0158	0.0238	0.0400	0.0210
4	0.0046	0.0043	0.0168	0.0243	0.0440	0.0220
5	0.0051	0.0044	0.0177	0.0250	0.0480	0.0240
7	0.0058	0.0048	0.0195	0.0275	0.0560	0.0260

Design AEP event	0.005	
spatial pattern =	3	Highlands
temporal pattern =	2	medium
Min 1-3 AEP	0.0044	

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0062	0.0047	0.0356	0.0045	0.1000	0.0525
1	0.0049	0.0045	0.0413	0.0058	0.1100	0.0475
2	0.0044	0.0047	0.0319	0.0050	0.0900	0.0500
3	0.0045	0.0049	0.0394	0.0050	0.1000	0.0525
4	0.0046	0.0051	0.0420	0.0051	0.1100	0.0550
5	0.0051	0.0052	0.0443	0.0053	0.1200	0.0600
7	0.0057	0.0056	0.0488	0.0058	0.1400	0.0650

Design AEP event	0.005	
spatial pattern =	1	Upper Cowlitz
temporal pattern =	3	long
Min 1-3 AEP	0.0033	

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0085	0.0060	0.0232	0.0273	0.0545	0.0255
1	0.0065	0.0046	0.0177	0.0307	0.0545	0.0218
2	0.0043	0.0038	0.0130	0.0227	0.0291	0.0182
3	0.0033	0.0034	0.0102	0.0148	0.0255	0.0145
4	0.0026	0.0030	0.0082	0.0102	0.0255	0.0109
5	0.0024	0.0026	0.0068	0.0080	0.0255	0.0091
7	0.0022	0.0024	0.0068	0.0080	0.0273	0.0091

Design AEP event	0.005	
spatial pattern =	3	Highlands
temporal pattern =	3	long
Min 1-3 AEP	0.0033	

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0085	0.0073	0.0510	0.0066	0.1200	0.0560
1	0.0064	0.0056	0.0390	0.0074	0.1200	0.0480
2	0.0043	0.0046	0.0285	0.0055	0.0640	0.0400
3	0.0033	0.0041	0.0225	0.0036	0.0560	0.0320
4	0.0026	0.0037	0.0180	0.0025	0.0560	0.0240
5	0.0023	0.0032	0.0150	0.0019	0.0560	0.0200
7	0.0022	0.0029	0.0150	0.0019	0.0600	0.0200

Design AEP event	0.005	
spatial pattern =	2	Toutle
temporal pattern =	1	short
Min 1-3 AEP	0.0038	

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0035	0.0100	0.0133	0.0020	0.0125	0.0146
1	0.0038	0.0100	0.0108	0.0032	0.0150	0.0156
2	0.0049	0.0131	0.0167	0.0042	0.0288	0.0208
3	0.0060	0.0144	0.0225	0.0046	0.0313	0.0260
4	0.0069	0.0163	0.0258	0.0052	0.0375	0.0323
5	0.0085	0.0181	0.0283	0.0060	0.0450	0.0375
7	0.0105	0.0219	0.0333	0.0072	0.0550	0.0479

Design AEP event	0.005	
spatial pattern =	4	Lowlands
temporal pattern =	1	short
Min 1-3 AEP	0.0035	

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0027	0.0185	0.0024	0.0058	0.0013	0.0043
1	0.0035	0.0185	0.0020	0.0092	0.0015	0.0046
2	0.0048	0.0242	0.0030	0.0121	0.0029	0.0062
3	0.0061	0.0265	0.0041	0.0133	0.0031	0.0077
4	0.0071	0.0300	0.0047	0.0150	0.0038	0.0095
5	0.0087	0.0335	0.0051	0.0173	0.0045	0.0111
7	0.0107	0.0404	0.0060	0.0208	0.0055	0.0142

Design AEP event	0.005	
spatial pattern =	2	Toutle
temporal pattern =	2	medium
Min 1-3 AEP	0.0044	

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0056	0.0107	0.0136	0.0031	0.0214	0.0188
1	0.0050	0.0102	0.0157	0.0046	0.0236	0.0170
2	0.0044	0.0107	0.0121	0.0037	0.0193	0.0179
3	0.0046	0.0113	0.0150	0.0037	0.0214	0.0188
4	0.0047	0.0116	0.0160	0.0038	0.0236	0.0196
5	0.0052	0.0118	0.0169	0.0040	0.0257	0.0214
7	0.0058	0.0129	0.0186	0.0046	0.0300	0.0232

Design AEP event	0.005	
spatial pattern =		uniform
temporal pattern =		flat
Min 1-3 AEP	0.0044	

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0053	0.0098	0.0098	0.0098	0.0098	0.0098
1	0.0048	0.0098	0.0098	0.0098	0.0098	0.0098
2	0.0044	0.0098	0.0098	0.0098	0.0098	0.0098
3	0.0044	0.0098	0.0098	0.0098	0.0098	0.0098
4	0.0043	0.0098	0.0098	0.0098	0.0098	0.0098
5	0.0046	0.0098	0.0098	0.0098	0.0098	0.0098
7	0.0046	0.0098	0.0098	0.0098	0.0098	0.0098

Design AEP event	0.005	
spatial pattern =	2	Toutle
temporal pattern =	3	long
Min 1-3 AEP	0.0034	

	Castle Rock	Jpper Cowlit:	Tilton	Toutle	Cas Locals	ResLoc
0	0.0070	0.0152	0.0230	0.0048	0.0274	0.0236
1	0.0062	0.0117	0.0176	0.0054	0.0274	0.0203
2	0.0044	0.0096	0.0128	0.0040	0.0174	0.0169
3	0.0034	0.0086	0.0101	0.0026	0.0160	0.0135
4	0.0027	0.0076	0.0081	0.0018	0.0160	0.0101
5	0.0024	0.0066	0.0068	0.0014	0.0160	0.0084
7	0.0022	0.0061	0.0068	0.0014	0.0167	0.0084

Design AEP event 0.002
spatial pattern = 1 Upper Cowlitz
temporal pattern = 1 short
Min 1-3 AEP 0.0015

	Castle Rock	Jpper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0017	0.0013	0.0080	0.0083	0.0173	0.0093
1	0.0015	0.0013	0.0065	0.0133	0.0192	0.0100
2	0.0018	0.0017	0.0100	0.0175	0.0295	0.0133
3	0.0023	0.0018	0.0135	0.0192	0.0313	0.0167
4	0.0027	0.0021	0.0155	0.0217	0.0360	0.0207
5	0.0033	0.0023	0.0170	0.0250	0.0416	0.0240
7	0.0042	0.0028	0.0200	0.0300	0.0491	0.0307

Design AEP event 0.002
spatial pattern = 3 Highlands
temporal pattern = 1 short
Min 1-3 AEP 0.0015

	Castle Rock	Jpper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0016	0.0020	0.0120	0.0014	0.0200	0.0140
1	0.0015	0.0020	0.0098	0.0023	0.0240	0.0150
2	0.0018	0.0027	0.0150	0.0030	0.0460	0.0200
3	0.0022	0.0029	0.0203	0.0033	0.0500	0.0250
4	0.0026	0.0033	0.0233	0.0037	0.0600	0.0310
5	0.0032	0.0037	0.0255	0.0043	0.0720	0.0360
7	0.0040	0.0044	0.0300	0.0051	0.0880	0.0460

Design AEP event 0.002
spatial pattern = 1 Upper Cowlitz
temporal pattern = 2 medium
Min 1-3 AEP 0.0017

	Castle Rock	Jpper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0026	0.0019	0.0057	0.0085	0.0160	0.0084
1	0.0019	0.0018	0.0066	0.0110	0.0176	0.0076
2	0.0017	0.0019	0.0051	0.0095	0.0144	0.0080
3	0.0017	0.0020	0.0063	0.0095	0.0160	0.0084
4	0.0018	0.0020	0.0067	0.0097	0.0176	0.0088
5	0.0020	0.0021	0.0071	0.0100	0.0192	0.0096
7	0.0022	0.0023	0.0078	0.0110	0.0224	0.0104

Design AEP event 0.002
spatial pattern = 3 Highlands
temporal pattern = 2 medium
Min 1-3 AEP 0.0017

	Castle Rock	Jpper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0026	0.0021	0.0143	0.0020	0.0400	0.0210
1	0.0019	0.0020	0.0165	0.0026	0.0440	0.0190
2	0.0017	0.0021	0.0128	0.0023	0.0360	0.0200
3	0.0017	0.0022	0.0158	0.0023	0.0400	0.0210
4	0.0018	0.0023	0.0168	0.0023	0.0440	0.0220
5	0.0020	0.0023	0.0177	0.0024	0.0480	0.0240
7	0.0022	0.0026	0.0195	0.0026	0.0560	0.0260

Design AEP event 0.002
spatial pattern = 1 Upper Cowlitz
temporal pattern = 3 long
Min 1-3 AEP 0.0012

	Castle Rock	Jpper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0037	0.0024	0.0064	0.0122	0.0400	0.0112
1	0.0026	0.0018	0.0049	0.0138	0.0400	0.0096
2	0.0016	0.0015	0.0036	0.0102	0.0213	0.0080
3	0.0012	0.0014	0.0028	0.0066	0.0187	0.0064
4	0.0010	0.0012	0.0023	0.0046	0.0187	0.0048
5	0.0009	0.0010	0.0019	0.0036	0.0187	0.0040
7	0.0008	0.0010	0.0019	0.0036	0.0200	0.0040

Design AEP event 0.002
spatial pattern = 3 Highlands
temporal pattern = 3 long
Min 1-3 AEP 0.0012

	Castle Rock	Jpper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0035	0.0031	0.0204	0.0028	0.0480	0.0224
1	0.0025	0.0024	0.0156	0.0032	0.0480	0.0192
2	0.0016	0.0020	0.0114	0.0023	0.0256	0.0160
3	0.0012	0.0018	0.0090	0.0015	0.0224	0.0128
4	0.0010	0.0016	0.0072	0.0011	0.0224	0.0096
5	0.0009	0.0014	0.0060	0.0008	0.0224	0.0080
7	0.0008	0.0012	0.0060	0.0008	0.0240	0.0080

Design AEP event 0.002
spatial pattern = 2 Toutle
temporal pattern = 1 short
Min 1-3 AEP 0.0015

	Castle Rock	Jpper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0014	0.0044	0.0058	0.0008	0.0055	0.0064
1	0.0015	0.0044	0.0047	0.0013	0.0065	0.0068
2	0.0019	0.0057	0.0073	0.0017	0.0125	0.0091
3	0.0023	0.0063	0.0098	0.0018	0.0136	0.0114
4	0.0026	0.0071	0.0113	0.0021	0.0164	0.0141
5	0.0032	0.0079	0.0124	0.0024	0.0196	0.0164
7	0.0040	0.0095	0.0145	0.0029	0.0240	0.0209

Design AEP event 0.002
spatial pattern = 4 Lowlands
temporal pattern = 1 short
Min 1-3 AEP 0.0014

	Castle Rock	Jpper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0011	0.0080	0.0010	0.0025	0.0005	0.0019
1	0.0014	0.0080	0.0008	0.0040	0.0006	0.0020
2	0.0019	0.0105	0.0012	0.0053	0.0012	0.0027
3	0.0024	0.0115	0.0016	0.0058	0.0013	0.0033
4	0.0028	0.0130	0.0019	0.0065	0.0015	0.0041
5	0.0034	0.0145	0.0020	0.0075	0.0018	0.0048
7	0.0041	0.0175	0.0024	0.0090	0.0022	0.0061

Design AEP event 0.002
spatial pattern = 2 Toutle
temporal pattern = 2 medium
Min 1-3 AEP 0.0017

	Castle Rock	Jpper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0023	0.0044	0.0056	0.0014	0.0088	0.0077
1	0.0019	0.0042	0.0065	0.0018	0.0097	0.0070
2	0.0017	0.0044	0.0050	0.0015	0.0079	0.0074
3	0.0017	0.0046	0.0062	0.0015	0.0088	0.0077
4	0.0017	0.0048	0.0066	0.0016	0.0097	0.0081
5	0.0019	0.0049	0.0069	0.0016	0.0106	0.0088
7	0.0021	0.0053	0.0076	0.0018	0.0124	0.0096

Design AEP event 0.002
spatial pattern = uniform
temporal pattern = flat
Min 1-3 AEP 0.0017

	Castle Rock	Jpper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0022	0.0042	0.0042	0.0042	0.0042	0.0042
1	0.0019	0.0042	0.0042	0.0042	0.0042	0.0042
2	0.0017	0.0042	0.0042	0.0042	0.0042	0.0042
3	0.0017	0.0042	0.0042	0.0042	0.0042	0.0042
4	0.0017	0.0042	0.0042	0.0042	0.0042	0.0042
5	0.0018	0.0042	0.0042	0.0042	0.0042	0.0042
7	0.0018	0.0042	0.0042	0.0042	0.0042	0.0042

Design AEP event 0.002
spatial pattern = 2 Toutle
temporal pattern = 3 long
Min 1-3 AEP 0.0013

	Castle Rock	Jpper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0029	0.0064	0.0097	0.0019	0.0129	0.0100
1	0.0025	0.0049	0.0074	0.0022	0.0129	0.0086
2	0.0017	0.0041	0.0054	0.0016	0.0069	0.0071
3	0.0013	0.0036	0.0043	0.0010	0.0060	0.0057
4	0.0010	0.0032	0.0034	0.0007	0.0060	0.0043
5	0.0009	0.0028	0.0029	0.0006	0.0060	0.0036
7	0.0008	0.0026	0.0029	0.0006	0.0064	0.0036

Design AEP event 0.001
spatial pattern = 1 Upper Cowlitz
temporal pattern = 1 short
Min 1-3 AEP 0.0007

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0008	0.0011	0.0024	0.0025	0.0052	0.0028
1	0.0007	0.0011	0.0020	0.0040	0.0058	0.0030
2	0.0009	0.0014	0.0030	0.0053	0.0088	0.0040
3	0.0011	0.0016	0.0041	0.0058	0.0094	0.0050
4	0.0013	0.0018	0.0047	0.0065	0.0108	0.0062
5	0.0016	0.0020	0.0051	0.0075	0.0125	0.0072
7	0.0020	0.0024	0.0060	0.0090	0.0147	0.0092

Design AEP event 0.001
spatial pattern = 3 Highlands
temporal pattern = 1 short
Min 1-3 AEP 0.0007

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0009	0.0011	0.0060	0.0008	0.0100	0.0070
1	0.0007	0.0011	0.0049	0.0012	0.0120	0.0075
2	0.0009	0.0014	0.0075	0.0016	0.0230	0.0100
3	0.0011	0.0016	0.0101	0.0018	0.0250	0.0125
4	0.0013	0.0018	0.0116	0.0020	0.0300	0.0155
5	0.0016	0.0020	0.0128	0.0023	0.0360	0.0180
7	0.0019	0.0024	0.0150	0.0028	0.0440	0.0230

Design AEP event 0.001
spatial pattern = 1 Upper Cowlitz
temporal pattern = 2 medium
Min 1-3 AEP 0.0008

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0013	0.0010	0.0029	0.0043	0.0080	0.0042
1	0.0009	0.0010	0.0033	0.0055	0.0088	0.0038
2	0.0008	0.0010	0.0026	0.0048	0.0072	0.0040
3	0.0008	0.0011	0.0032	0.0048	0.0080	0.0042
4	0.0008	0.0011	0.0034	0.0049	0.0088	0.0044
5	0.0009	0.0011	0.0035	0.0050	0.0096	0.0048
7	0.0010	0.0012	0.0039	0.0055	0.0112	0.0052

Design AEP event 0.001
spatial pattern = 3 Highlands
temporal pattern = 2 medium
Min 1-3 AEP 0.0008

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0014	0.0012	0.0071	0.0011	0.0200	0.0105
1	0.0010	0.0011	0.0083	0.0015	0.0220	0.0095
2	0.0008	0.0012	0.0064	0.0013	0.0180	0.0100
3	0.0008	0.0012	0.0079	0.0013	0.0200	0.0105
4	0.0009	0.0013	0.0084	0.0013	0.0220	0.0110
5	0.0010	0.0013	0.0089	0.0013	0.0240	0.0120
7	0.0010	0.0014	0.0098	0.0015	0.0280	0.0130

Design AEP event 0.001
spatial pattern = 1 Upper Cowlitz
temporal pattern = 3 long
Min 1-3 AEP 0.0006

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0018	0.0013	0.0032	0.0061	0.0200	0.0056
1	0.0013	0.0010	0.0024	0.0069	0.0200	0.0048
2	0.0008	0.0008	0.0018	0.0051	0.0107	0.0040
3	0.0006	0.0007	0.0014	0.0033	0.0093	0.0032
4	0.0005	0.0006	0.0011	0.0023	0.0093	0.0024
5	0.0004	0.0006	0.0009	0.0018	0.0093	0.0020
7	0.0004	0.0005	0.0009	0.0018	0.0100	0.0020

Design AEP event 0.001
spatial pattern = 3 Highlands
temporal pattern = 3 long
Min 1-3 AEP 0.0006

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0019	0.0017	0.0102	0.0015	0.0240	0.0112
1	0.0013	0.0013	0.0078	0.0017	0.0240	0.0096
2	0.0008	0.0011	0.0057	0.0013	0.0128	0.0080
3	0.0006	0.0010	0.0045	0.0008	0.0112	0.0064
4	0.0005	0.0009	0.0036	0.0006	0.0112	0.0048
5	0.0004	0.0007	0.0030	0.0005	0.0112	0.0040
7	0.0004	0.0007	0.0030	0.0005	0.0120	0.0040

Design AEP event 0.001
spatial pattern = 2 Toutle
temporal pattern = 1 short
Min 1-3 AEP 0.0007

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0007	0.0022	0.0029	0.0005	0.0027	0.0032
1	0.0007	0.0022	0.0024	0.0008	0.0033	0.0034
2	0.0009	0.0029	0.0036	0.0011	0.0063	0.0045
3	0.0011	0.0031	0.0049	0.0012	0.0068	0.0057
4	0.0013	0.0035	0.0056	0.0014	0.0082	0.0070
5	0.0016	0.0040	0.0062	0.0016	0.0098	0.0082
7	0.0019	0.0048	0.0073	0.0019	0.0120	0.0105

Design AEP event 0.001
spatial pattern = 4 Lowlands
temporal pattern = 1 short
Min 1-3 AEP 0.0007

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0005	0.0041	0.0005	0.0013	0.0003	0.0010
1	0.0007	0.0041	0.0004	0.0021	0.0003	0.0010
2	0.0009	0.0054	0.0006	0.0027	0.0006	0.0014
3	0.0012	0.0059	0.0008	0.0030	0.0006	0.0017
4	0.0013	0.0067	0.0009	0.0034	0.0008	0.0021
5	0.0016	0.0075	0.0010	0.0039	0.0009	0.0025
7	0.0020	0.0091	0.0012	0.0047	0.0011	0.0032

Design AEP event 0.001
spatial pattern = 2 Toutle
temporal pattern = 2 medium
Min 1-3 AEP 0.0008

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0012	0.0022	0.0028	0.0009	0.0044	0.0039
1	0.0009	0.0021	0.0032	0.0011	0.0049	0.0035
2	0.0008	0.0022	0.0025	0.0010	0.0040	0.0037
3	0.0008	0.0023	0.0031	0.0010	0.0044	0.0039
4	0.0008	0.0024	0.0033	0.0010	0.0049	0.0040
5	0.0009	0.0024	0.0035	0.0010	0.0053	0.0044
7	0.0010	0.0026	0.0038	0.0011	0.0062	0.0048

Design AEP event 0.001
spatial pattern = uniform
temporal pattern = flat
Min 1-3 AEP 0.0008

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0011	0.0022	0.0022	0.0022	0.0022	0.0022
1	0.0009	0.0022	0.0022	0.0022	0.0022	0.0022
2	0.0008	0.0022	0.0022	0.0022	0.0022	0.0022
3	0.0008	0.0022	0.0022	0.0022	0.0022	0.0022
4	0.0008	0.0022	0.0022	0.0022	0.0022	0.0022
5	0.0009	0.0022	0.0022	0.0022	0.0022	0.0022
7	0.0008	0.0022	0.0022	0.0022	0.0022	0.0022

Design AEP event 0.001
spatial pattern = 2 Toutle
temporal pattern = 3 long
Min 1-3 AEP 0.0006

	Castle Rock	Upper Cowlitz	Tilton	Toutle	Cas Locals	ResLoc
0	0.0014	0.0032	0.0049	0.0010	0.0064	0.0050
1	0.0012	0.0025	0.0037	0.0011	0.0064	0.0043
2	0.0008	0.0020	0.0027	0.0008	0.0034	0.0036
3	0.0006	0.0018	0.0021	0.0005	0.0030	0.0029
4	0.0005	0.0016	0.0017	0.0004	0.0030	0.0021
5	0.0004	0.0014	0.0014	0.0003	0.0030	0.0018
7	0.0004	0.0013	0.0014	0.0003	0.0032	0.0018

Appendix C. Cowlitz River Hydraulic Model

C.1. Modeling Approach

Stage-discharge curves used in the Level of Protection update are developed using the Corps of Engineer's computer model HEC-RAS (River Analysis System), Version 4.0. Cross Section data used to describe river geometry was obtained from a combination of LiDAR surveys, bathymetric surveys, and field investigations. Measured high water marks and gaged stage data from a hydrologic event in January 2009 were used to calibrate the model parameters, specifically the Manning's roughness coefficients. In developing the modeling approach for the Cowlitz River during the level of protection analysis for related levees, it was recognized that the hydraulic models used for this analysis should represent only current conditions. Estimated effects of future sedimentation or predicted changes in cross section geometry are not included in this analysis. Twenty-three steady state frequency events were modeled to describe the stage discharge rating curve at each levee index point. This appendix describes the details of the various components of the hydraulic model used to generate stage discharge curves for the 2009 Level of Protection analysis.

C.2. Data sources

The steady state one-dimensional fixed bed hydraulic model used in this level of protection analysis utilized 101 cross sections and seven river crossings to describe the lower 20 miles of the Cowlitz River. Information used to describe the geometry of these features was gathered from a variety of sources. Table C. 1 provides an inventory of the data sources for each cross section in the hydraulic model. For all cross sections, information describing channel geometry, which was updated based on a 2009 hydrosurvey, was merged with overbank geometry derived from a variety of sources. Prior to merging cross section geometry data, consistent horizontal and vertical control was assured. Horizontal datum was consistently based on state plane projected coordinate system for Washington South and the vertical datum was based on NAVD 88.

Table C. 1. Cross Section Geometry Data Sources for the Cowlitz River Hydraulic Model

River Mile	**Coverage**		**River Mile**	**Coverage**	
	Overbanks	**Channel**		**Overbank**	**Channel**
20.06	2007 LiDAR[1]	2009 Hydrosurvey[4]	7.81	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
19.52	2007 LiDAR[1]	2009 Hydrosurvey[4]	7.68	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
19.05	2007 LiDAR[1]	2009 Hydrosurvey[4]	7.59	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
18.69	2007 LiDAR[1]	2009 Hydrosurvey[4]	7.47	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
18.11	2007 LiDAR[1]	2009 Hydrosurvey[4]	7.3	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
17.66	2007 LiDAR[1]	2009 Hydrosurvey[4]	7.12	2007 LiDAR[1]	2009 Hydrosurvey[4]
17.42	2007 LiDAR[1]	2009 Hydrosurvey[4]	7	2007 LiDAR[1]	2009 Hydrosurvey[4]
17.05	2007 LiDAR[1]	2009 Hydrosurvey[4]	6.78	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
17	2007 LiDAR[1]	2009 Hydrosurvey[4]	6.75	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
16.64	2007 LiDAR[1]	2009 Hydrosurvey[4]	6.41	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
16.4	2007 LiDAR[1]	2009 Hydrosurvey[4]	6.19	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
16.1	2007 LiDAR[1]	2009 Hydrosurvey[4]	5.94	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
15.91	2007 LiDAR[1]	2009 Hydrosurvey[4]	5.69	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
15.59	2007 LiDAR[1]	2009 Hydrosurvey[4]	5.57	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
15.33	2007 LiDAR[1]	2009 Hydrosurvey[4]	5.37	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
15.05	2007 LiDAR[1]	2009 Hydrosurvey[4]	5.23	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
14.78	2007 LiDAR[1]	2009 Hydrosurvey[4]	5.2	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
14.51	2007 LiDAR[1]	2009 Hydrosurvey[4]	5.09	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
14.11	2007 LiDAR[1]	2009 Hydrosurvey[4]	5.05	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
13.65	2007 LiDAR[1]	2009 Hydrosurvey[4]	4.9	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
13.46	2007 LiDAR[1]	2009 Hydrosurvey[4]	4.68	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
13.25	2007 LiDAR[1]	2009 Hydrosurvey[4]	4.47	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
13.06	2007 LiDAR[1]	2009 Hydrosurvey[4]	4.25	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
12.87	2007 LiDAR[1]	2009 Hydrosurvey[4]	4.02	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
12.65	2007 LiDAR[1]	2009 Hydrosurvey[4]	3.8	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
12.4	2007 LiDAR[1]	2009 Hydrosurvey[4]	3.7	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
12.23	2007 LiDAR[1]	2009 Hydrosurvey[4]	3.59	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
12.01	2007 LiDAR[1]	2009 Hydrosurvey[4]	3.41	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]

River Mile	Coverage		River Mile	Coverage	
	Overbanks	**Channel**		**Overbank**	**Channel**
11.83	2007 LiDAR[1]	2009 Hydrosurvey[4]	3.27	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
11.65	2007 LiDAR[1]	2009 Hydrosurvey[4]	3.15	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
11.55	2007 LiDAR[1]	2009 Hydrosurvey[4]	3.06	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
11.03	2007 LiDAR[1]	2009 Hydrosurvey[4]	2.91	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
10.57	2007 LiDAR[1]	2009 Hydrosurvey[4]	2.78	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
10.3	2007 LiDAR[1]	2009 Hydrosurvey[4]	2.54	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
10.07	2007 LiDAR[1]	2009 Hydrosurvey[4]	2.31	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
9.88	2007 LiDAR[1]	2009 Hydrosurvey[4]	1.99	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
9.53	2007 LiDAR[1]	2009 Hydrosurvey[4]	1.71	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
9.4	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	1.61	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
9.07	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	1.57	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]
8.83	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	1.38	PSLC LiDAR[3]	2009 Hydrosurvey[4]
8.64	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	1.34	PSLC LiDAR[3]	2009 Hydrosurvey[4]
8.48	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	1.12	PSLC LiDAR[3]	2009 Hydrosurvey[4]
8.39	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	0.88	PSLC LiDAR[3]	2009 Hydrosurvey[4]
8.3	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	0.67	PSLC LiDAR[3]	2009 Hydrosurvey[4]
8.23	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	0.41	PSLC LiDAR[3]	2009 Hydrosurvey[4]
8.11	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	0.18	PSLC LiDAR[3]	2009 Hydrosurvey[4]
8.07	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	0.01	PSLC LiDAR[3]	2009 Hydrosurvey[4]
8.01	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]			

[1] Aerial LiDAR flown by USACE in 2007 (average accuracy 0.28 ft)

[2] USGS 10 meter DEM of Mount St. Helens and Vicinity

[3] Puget Sound Lidar Consortium: 2005 Lower Columbia River (30 cm accuracy)

[4] August 2009 Hydrosurvey: 3 ft horizontal accuracy ; 0.5 ft verticle accuracy

Seven bridge crossings were included in the 20 mile section of the lower Cowlitz River modeled in the hydraulic model. Various sources of information were compiled to create the necessary input for the bridge crossings. From the 2007 LiDAR information, two rasters were extracted to represent the *bare earth* or ground elevations and the *highest hit* or the elevation of the first object hit by the laser pulse. Deck roadway data was typically obtained from the portion of the 2007 LiDAR data that included the highest hit objects. Cross section data utilized the 2007 bare earth LiDAR data where coverage permitted. Outside of the LiDAR coverage, cross section data was obtained from either a Mount St. Helens DEM created by the USGS or from a DEM created by the Puget Sound Lidar Consortium (PSLC). Pier locations for the bridge were obtained from aerial photography. Pier sizes were measured from field reconnaissance. Table C. 2 summarizes the sources of data utilized in creating bridge geometry.

Table C. 2. Bridge Data Sources for the Cowlitz River Hydraulic Model

River Mile	Description	Overbanks	Channel	Deck Roadway	Pier Locations
17.03	Castle Rock Bridge	2007 LiDAR[1]	2009 Hydrosurvey[4]	2007 LiDAR[5]	Est. from Aerial Photography
8.09	Lexington Bridge	2007 LiDAR[1]	2009 Hydrosurvey[4]	2007 LiDAR[5]	Est. from Aerial Photography
6.76	Beacon Hill R/R Bridge	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	Rail road deck taken from left and right abutment elevations	Est. from Aerial Photography
5.21	Highway 411 Bridge	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	2007 LiDAR[5]	Est. from Aerial Photography
5.07	Main St Bridge	10m DEM[2] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	2007 LiDAR[5]	Est. from Aerial Photography
1.59	432 Bridge	PSLC LiDAR[3] and 2007 LiDAR[1]	2009 Hydrosurvey[4]	2007 LiDAR[5]	Est. from Aerial Photography
1.35	Longveiw R/R Bridge	PSLC LiDAR[3]	2009 Hydrosurvey[4]	2007 LiDAR[5]	Est. from Aerial Photography

[1] Aerial LiDAR flown by USACE in 2007 (average accuracy 0.28 ft)
[2] USGS 10 meter DEM of Mount St. Helens and Vicinity
[3] Puget Sound Lidar Consortium: 2005 Lower Columbia River (30 cm accuracy)
[4] August 2009 Hydrosurvey: 3 ft horizontal accuracy ; 0.5 ft verticle accuracy
[5] High hit LiDAR data was used for Deck/Roadway data

C.3. EXPANSION/CONTRACTION COEFFICIENTS

The downstream 20 miles of the Cowlitz River is highly leveed and contains very few to no contractions or expansions that require adjustments to the default values. One apparent exception would be the Castle Rock Bridge at approximately RM 17. A contraction exists when flows become out of bank, but the effect

diminishes during very high flows with water flowing over the roadway on the right overbank. The calibration event was roughly a 40-year event when the apparent contraction is occurring at the Castle Rock bridge, however the model did not require adjustment of the expansion/contraction coefficients beyond their defaults to calibrate. For this reason expansion and contraction coefficients are set at their default values of 0.3 and 0.1 respectively.

C.4. BOUNDARY CONDITIONS FOR WATER SURFACE PROFILES

Boundary conditions required to simulate the Cowlitz River water surface profiles consist of river discharges at the upstream limit of the model at the Toutle River confluence, downstream tributary inflows and starting water surface elevations at the confluence of the Cowlitz and Columbia Rivers. 23 discharge frequency flows were prepared as inputs to the steady state HEC-RAS hydraulic model and were used to produce the stage discharge rating curves required for the LOP assessment model (HEC-FDA). Discharge inputs at the upstream boundary and tributaries are based on the flow frequency curves generated from the hydrologic analysis outlined in Appendix B. Flow rates for the 23 profiles are summarized in Table C. 3.

Table C. 3. Flow Rates used in the Cowlitz River Hydraulic Model

Frequency Event	Percent Chance Exceedance	Cowlitz River Peak Flow (cfs)			
		at Castle Rock RM 20.06	below Arkansas Cr. RM 16.1	below Ostrander Cr. RM 8.64	below Coweeman R. RM 1.61
1.00	99.99	10,000	10,200	10,400	11,200
1.01	99	11,000	11,500	11,700	13,300
1.05	95	18,000	18,600	19,000	21,100
1.11	90	24,000	24,700	25,200	27,700
1.25	80	32,000	32,900	33,400	36,400
1.43	70	36,500	37,500	38,100	41,500
1.67	60	41,000	42,200	42,800	46,600
2.00	50	46,000	47,300	48,000	52,200
2.50	40	51,000	52,400	53,200	57,800
3.33	30	58,000	59,600	60,500	65,600
5	20	66,000	67,800	68,800	74,500
10	10	80,000	82,200	83,400	90,100
20	5	96,000	98,600	99,900	107,500
25	4	100,000	102,700	104,100	112,000
50	2	108,000	111,000	112,600	121,300
100	1	113,000	116,400	118,200	127,700
143	0.7	117,000	120,600	122,400	132,400
200	0.5	124,000	127,700	129,700	140,000
500	0.2	160,000	164,200	166,500	177,800
1000	0.1	190,000	194,600	197,000	209,100
1250	0.08	210,000	214,700	217,100	229,500
2000	0.05	300,000	304,900	307,500	320,400
10000	0.01	390,000	395,800	398,800	413,500

Downstream boundary inputs were developed from Columbia River stage frequency data at the confluence with the Cowlitz River. Columbia River flood profiles, which were used to determine the starting water surface elevations for the Cowlitz River hydraulic model, were developed in study of the Columbia River basin by Corps of Engineers in 1987. Flood profiles along the Columbia River used in determining the starting water surface elevations for the Cowlitz model are shown in Figure C. 1.

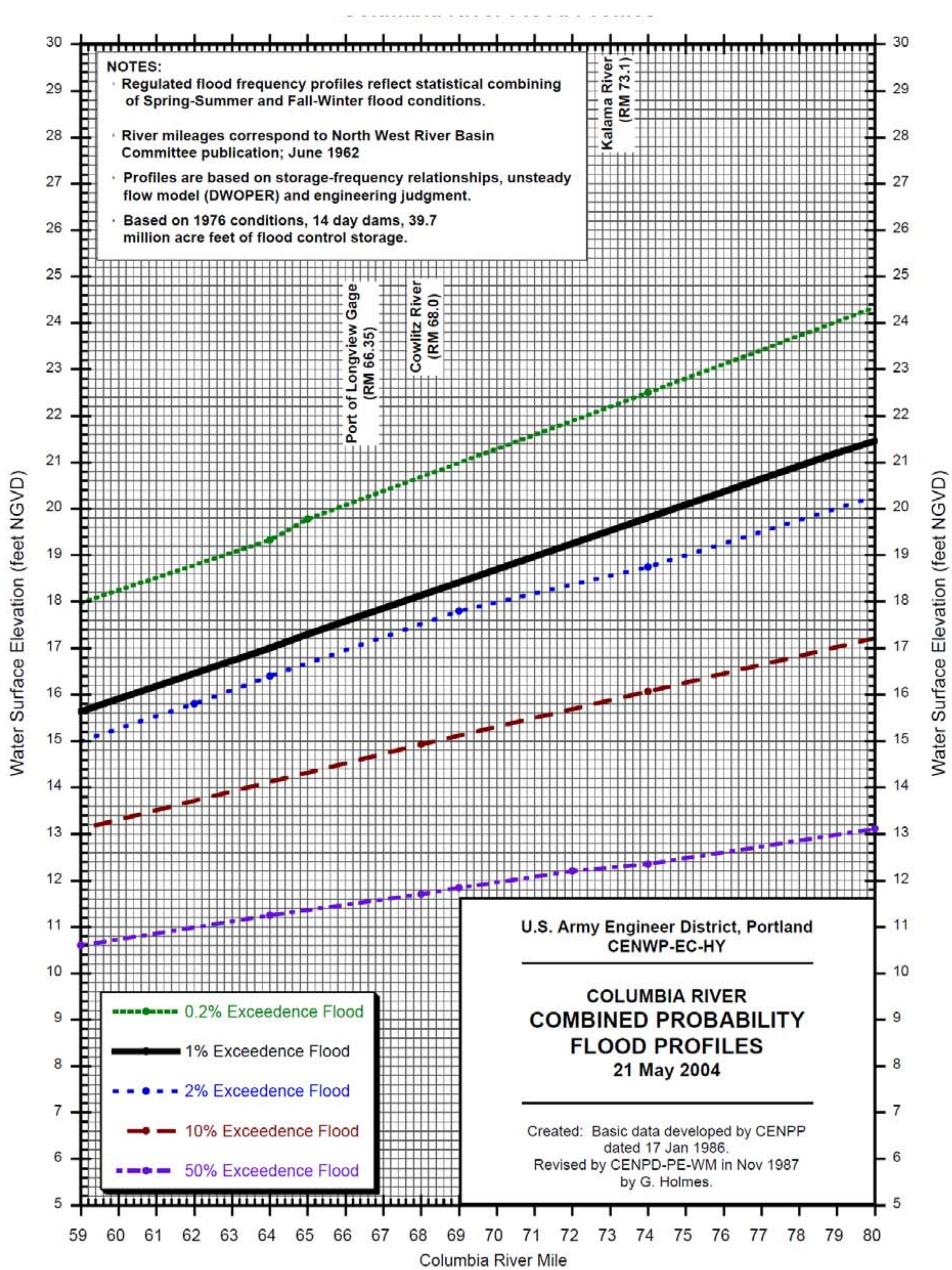


Figure C. 1. Columbia River Flood Profiles

Figure C. 1 only shows five (5) exceedance flood profiles. Starting water surface elevations for the 23 profiles modeled in the Cowlitz River hydraulic model were interpolated (with exceedance probability in a log scale) from Figure C. 1. It should be noted that the Columbia River flood profiles shown in Figure C. 1 are based on the NGVD 1929 and were converted to NAVD 88 using the Corps of Engineers conversion utility Corpscon (Version 6.0.1).

Table C. 4. Stage Frequency for Downstream Boundary Conditions on Cowlitz River

Frequency Event	Percent Chance Exceedance	Starting Water Surface for Cowlitz Hydraulic Model (ft NAVD 88)
1.00	0.9999	10.81
1.01	0.99	11.05
1.05	0.95	12.05
1.11	0.9	12.65
1.25	0.8	13.45
1.43	0.7	13.90
1.67	0.6	14.43
2.00	0.5	15.05
2.50	0.4	15.57
3.33	0.3	16.25
5	0.2	17.05
10	0.1	18.27
20	0.05	19.45
25	0.04	19.80
50	0.02	20.87
100	0.01	21.50
143	0.007	22.07
200	0.005	22.60
500	0.002	24.05
1000	0.001	25.05
1250	0.0008	25.37
2000	0.0005	26.05
10000	0.0001	28.37

For the purposes of this level of protection analysis the starting water surface elevations were chosen based on a coincident peak along the Columbia River at the mouth of the Cowlitz River. This assumption provides a conservative estimation of the flood profiles along the lower Cowlitz River and has been made in all previous LOP assessments. A coincident peak analysis would likely improve level of protection results for the levees in the lower end of the system, but would likely not substantially effect the level of protection of the more critical levees in the upstream end of the study reach.

C.5. Model Calibration and Manning's Roughness

C.5.1. Introduction

Calibration of the Cowlitz River hydraulic model consisted of adjustment of the Manning's roughness values until the resulting computed water surface profile matched observed data obtained from a flood event that occurred in January 2009. The August 2009 hydrosurvey was used to represent the most current channel conditions. Peak discharge during this flooding event was based on data from the USGS Castle Rock gage located downstream of the Castle Rock Bridge crossing at approximately river mile 17. Based on updated flood frequency flows, a frequency was assigned to the measured peak at Castle Rock. Contribution from the tributaries during this event was based on the flood frequency curves relative to the flood frequency determined from the Castle Rock gage data and an assumption of non-coincidence described in detail in Appendix B. After the January flood event, high water marks were identified and surveyed by Corps of Engineers personnel. Surveyed high water marks combined with the recording stage gage data along the Cowlitz combine to provide a observed dataset of 11 points to which the hydraulic model was calibrated. The calibration effort for the hydraulic model was tailored for the LOP estimate specifically. For the LOP, the focus is on the high flows primarily concerned with the frequency events near failure. Calibration of low flow conditions, or varying roughness values based on discharge, was considered inappropriate for this application. The calibration effort for the hydraulic model was tailored for the LOP estimate specifically. For the LOP, the focus is on the high flows primarily concerned with the frequency events near failure. Calibration of low flow conditions, or varying roughness values based on discharge, was considered inappropriate for this application. The following sub sections discuss the data used to calibrate the hydraulic model.

C.5.2. Calibration Boundary Conditions

Upstream boundary conditions for the steady state calibration model was obtained from records of the January 2009 event gathered by the USGS gage at Castle Rock (gage number 14243000). A peak discharge of 106,000 cfs was observed in the recorded hydrograph at 7:45 am on January 8, 2009. Figure C. 2 shows a portion of the recorded flow record from the Castle Rock gage, including the January 2009 calibration event.

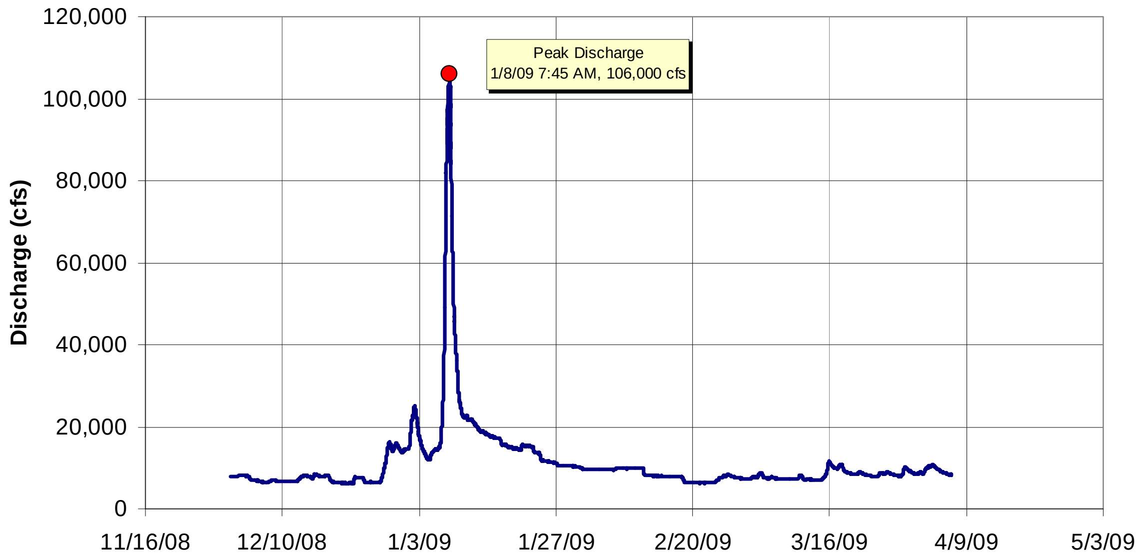


Figure C. 2. Flood Hydrograph at Castle Rock for the January 2009 Event

Using the flood frequency curves developed from the 2009 updated hydrologic analysis presented in Appendix B, it was estimated that the January 2009 event represented an event with a frequency of approximately 0.025 or a 40-year event. Interpolation of the January 2009 event from the updated flood frequency curve is shown graphically in Figure C. 3.

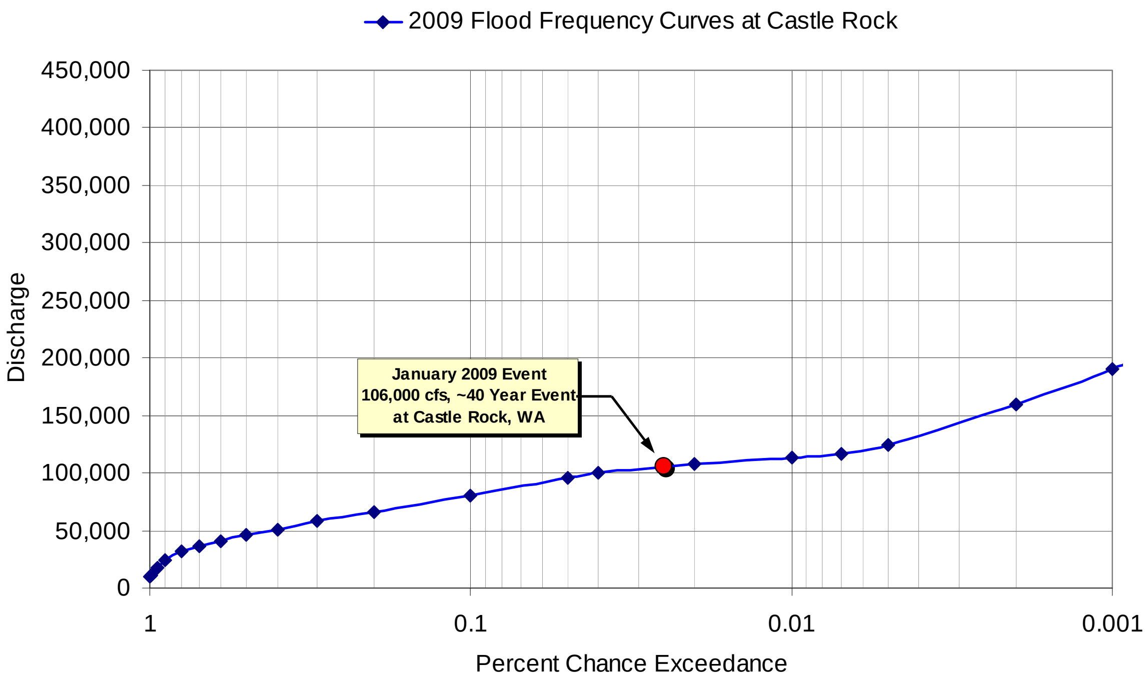


Figure C. 3. Frequency of Calibration Event in January 2009

Contributions of the Arkansas Creek, Ostrander Creek, and Coweeman River for the calibration model were computed from the corresponding flood frequency discharges shown in Table C. 3. Table C. 5 shows the discharges used in the steady state calibration model.

Table C. 5. Calibration Hydrology for the Cowlitz River

Discharge Input Location	Cowlitz River Mile	Discharge (cfs)
Cowlitz @ Castle Rock	20.06	106,000
Cowlitz below Arkansas River	16.10	108,925
Cowlitz below Ostrander Creek	8.64	110,475
Cowlitz below Coweeman River	1.61	118,975

Downstream starting water surface elevations for the calibration event were taken from a USGS gage at Longview, WA (gage no. 14246099) where stage data was obtained for a period corresponding to the January 2009 event. The peak stage of 15.69, which occurred at 12:41 pm on 1/8/09, was used as the downstream boundary condition for the steady state calibration model. Depiction of the stage data relative to the selected starting water surface elevation is shown in Figure C. 4.

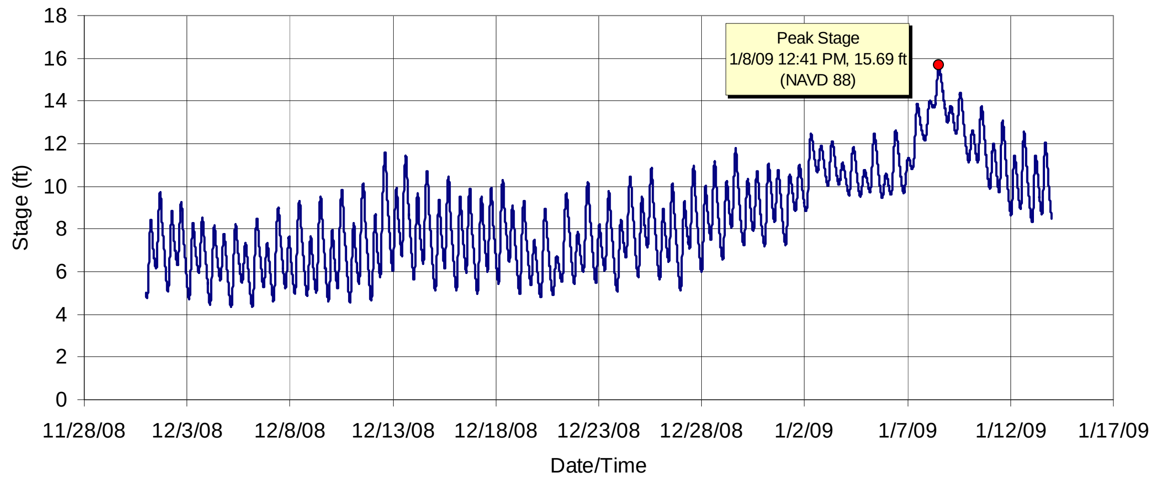


Figure C. 4. Stage Data for the January 2009 from the Columbia River gage at Longview, WA

A single profile steady state model was created using the discharges shown in Table C. 4 and the starting water surface shown in Figure C. 4. Computed water surface profiles from the model created using these boundary conditions were compared to observed data collected in January 2009.

C.5.3. Observed Data

Observed data collected in the month of January 2009 consisted of high water marks that were identified and surveyed immediately after flooding subsided along the Cowlitz and from gages which recorded reliable stage information throughout the flood event. Location of these observed points is shown on Figure 1-2.

A total of 11 observed water surface elevations, two of which were obtained from recorded stage gage data at the Castle Rock gage and at the Schmidt gage and 9 points were obtained from high water surveys conducted after the January 2009 event.

Table C. 6. Observed Points used in Steady State Calibration

Observed Point	High Water Elevation (ft, NAVD)	River Mile	Data Source
Gerhardt Gardens Gage	17.76	1.71	HWM[1]
Golf Course	20.42	2.78	HWM[1]
Upstream of Kelso Bridge	25.8	5.94	HWM[1]
Ostrander Gage	30.89	7.3	HWM[1]
Lexington Park	35.54	9.07	HWM[1]
Lexington Gage	37.42	9.53	HWM[1]
Overbank, near Sandy Bend Road	44.08	12.4	HWM[1]
Horseshoe Bend Point	45.11	12.87	HWM[1]
Castle Rock Gage	54.64	17	Gage Data
North end of Castle Rock	56.74	17.42	HWM[1]
Schmidt Gage	63	19.52	Gage Data

[1]HWM: High Water Mark surveyed after Jan 2009 Storm Event

C.5.4. Calibrated Manning's Roughness

Computed water surface profiles of the January 2009 event were compared to the observed data presented in Table C. 6. In order to produce a water surface profile that accurately predicted the observed data, Manning's roughness coefficients were adjusted accordingly. Since the observed data are spread fairly evenly from the downstream to the upstream ends of the model, the resulting Manning's roughness values are representative of the hydraulic conditions present in the Cowlitz River. Table C. 7 summarizes the roughness values that were used to most accurately predict the observed water surface profiles.

Table C. 7. Calibrated Manning's Roughness Values

River Station	Left Overbank	Channel	Right Overbank	River Station	Left Overbank	Channel	Right Overbank
20.06	0.07	0.035	0.07	7.68	0.07	0.019	0.07
19.52	0.07	0.035	0.07	7.59	0.07	0.019	0.07
19.05	0.07	0.035	0.07	7.47	0.07	0.019	0.07
18.69	0.07	0.035	0.07	7.3	0.07	0.019	0.07
18.11	0.07	0.035	0.07	7.12	0.07	0.019	0.07
17.66	0.07	0.035	0.07	7	0.07	0.019	0.07
17.42	0.07	0.035	0.07	6.78	0.07	0.019	0.07
17.05	0.07	0.035	0.07	6.76	Beacon Hill R/R Bridge		
17.03	Castle Rock Bridge			6.75	0.07	0.015	0.07
17	0.07	0.032	0.07	6.41	0.07	0.015	0.07
16.64	0.07	0.032	0.07	6.19	0.07	0.015	0.07
16.4	0.07	0.032	0.07	5.94	0.07	0.015	0.07
16.1	0.07	0.032	0.07	5.69	0.07	0.015	0.07
15.91	0.07	0.032	0.07	5.57	0.07	0.015	0.07
15.59	0.07	0.032	0.07	5.37	0.07	0.015	0.07
15.33	0.07	0.032	0.07	5.23	0.07	0.015	0.07
15.05	0.07	0.032	0.07	5.21	Highway 411 Bridge		
14.78	0.07	0.032	0.07	5.2	0.07	0.015	0.07
14.51	0.07	0.032	0.07	5.09	0.07	0.015	0.07
14.11	0.07	0.032	0.07	5.07	Main Street Bridge		
13.65	0.07	0.032	0.07	5.05	0.07	0.015	0.07
13.46	0.07	0.032	0.07	4.9	0.07	0.015	0.07
13.25	0.07	0.032	0.07	4.68	0.07	0.015	0.07
13.06	0.07	0.032	0.07	4.47	0.07	0.015	0.07
12.87	0.07	0.032	0.07	4.25	0.07	0.015	0.07
12.65	0.07	0.032	0.07	4.02	0.07	0.015	0.07
12.4	0.07	0.032	0.07	3.8	0.07	0.015	0.07
12.23	0.07	0.032	0.07	3.7	0.07	0.015	0.07
12.01	0.07	0.032	0.07	3.59	0.07	0.015	0.07
11.83	0.07	0.028	0.07	3.41	0.07	0.015	0.07
11.65	0.07	0.028	0.07	3.27	0.07	0.015	0.07

11.55	0.07	0.028	0.07
11.03	0.07	0.028	0.07
10.57	0.07	0.028	0.07
10.3	0.07	0.028	0.07
10.07	0.07	0.028	0.07
9.88	0.07	0.028	0.07
9.53	0.07	0.022	0.07
9.4	0.07	0.022	0.07
9.07	0.07	0.022	0.07
8.83	0.07	0.022	0.07
8.64	0.07	0.022	0.07
8.48	0.07	0.022	0.07
8.39	0.07	0.022	0.07
8.3	0.07	0.022	0.07
8.23	0.07	0.022	0.07
8.11	0.07	0.022	0.07
8.09	Lexiington Bridge		
8.07	0.07	0.019	0.07
8.01	0.07	0.019	0.07
7.81	0.07	0.019	0.07

3.15	0.07	0.015	0.07
3.06	0.07	0.015	0.07
2.91	0.07	0.015	0.07
2.78	0.07	0.015	0.07
2.54	0.07	0.015	0.07
2.31	0.07	0.015	0.07
1.99	0.07	0.015	0.07
1.71	0.07	0.015	0.07
1.61	0.07	0.015	0.07
1.59	Highway 432 Bridge		
1.57	0.07	0.015	0.07
1.38	0.07	0.015	0.07
1.35	Longview R/R Bridge		
1.34	0.07	0.015	0.07
1.12	0.07	0.015	0.07
0.88	0.07	0.015	0.07
0.67	0.07	0.015	0.07
0.41	0.07	0.015	0.07
0.18	0.07	0.015	0.07
0.01	0.07	0.015	0.07

C.5.5. Manning's Roughness Justification

At the onset of the 2009 LOP investigation, the approach to determining a suitable Manning's roughness value for the Cowlitz River was divided into three distinct efforts. First, the Manning's roughness is adjusted based on various calibration techniques. Secondly, the range of Manning's roughness values are computed from predictions of the regime and theory regarding velocity profiles. And finally, roughness values from previous LOP studies are used were compared to current roughness estimates to provide a sense of the overall variation in roughness. Each method is address individually in the following paragraphs. Ultimately, the Manning's roughness values selected for the Cowlitz River hydraulic model were intended for use with the LOP study only, where the main concern is estimating rating curves that are to be used in evaluating the probability of levee failure where fragility curves only vary at high stages and high flows.

Regime change in terms of the 2009 LOP analysis is a characteristic that is used to describe changes to the bedform in a sand bed channel, where ripples, dunes, washed out dunes, and antidunes are possible. It is not, however, appropriate to describe a gravel bed channel in terms of the same type of bedforms which are present in the sand bed channel. In the case of the lower Cowlitz, there is a fining trend that occurs from the mouth of the Toutle River where a significant gravel component is present to the Columbia River where the channel consists of mainly sand. Indeed a clear distinction between upper and lower end of the Cowlitz River can be made at roughly RM 10. This distinction is based on changes in channel planform and bed material size. **Error! Reference source not found.** shows a plot of D50 over time with respect to river mile for the Lower Cowlitz River.

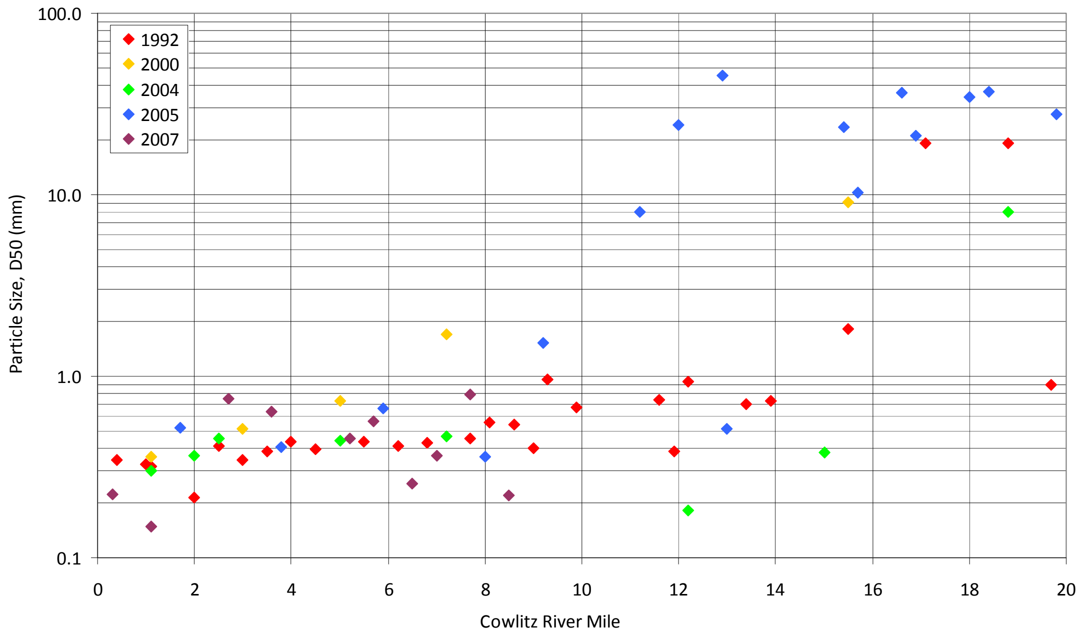


Figure C. 5. D50 Particle Size along the Cowlitz River

As can be seen in Figure C. 5, a discontinuity can be roughly seen at about river mile 10 where the D50 increase substantially. Field observations confirm this discontinuity and as a result, for the purposes of describing the Cowlitz system in the LOP analysis, RM 10 is used as a pivot point below which bed forms that are characteristic of sand bed channels are possible and above which gravel bed mechanics are more appropriate.

As discussed in the LOP report, an effort was made to predict the bedform associated with the Cowlitz River using the Van Rijn's bed form prediction method (Julien, 1998). Results from this method indicate that the bedform reaches transition to upper regime around the 10 percent AEP (~70,000 cfs) below RM 7. For more frequent events the bedform is generally in lower regime dunes or plane bed. However, the probability of levee failure below 10 percent AEP is zero and varying the roughness by discharge will not effect the overall LOP estimate. The focus was therefore centered on refining the estimate of the roughness coefficient for the larger flood events. Upper regime bedform classification was corroborated with a second bedform predictor that relates depth with Froude number. Figure C. 6 shows the relationship between the Froude number and depth developed from a large number of laboratory measurements (Julien, 1998).

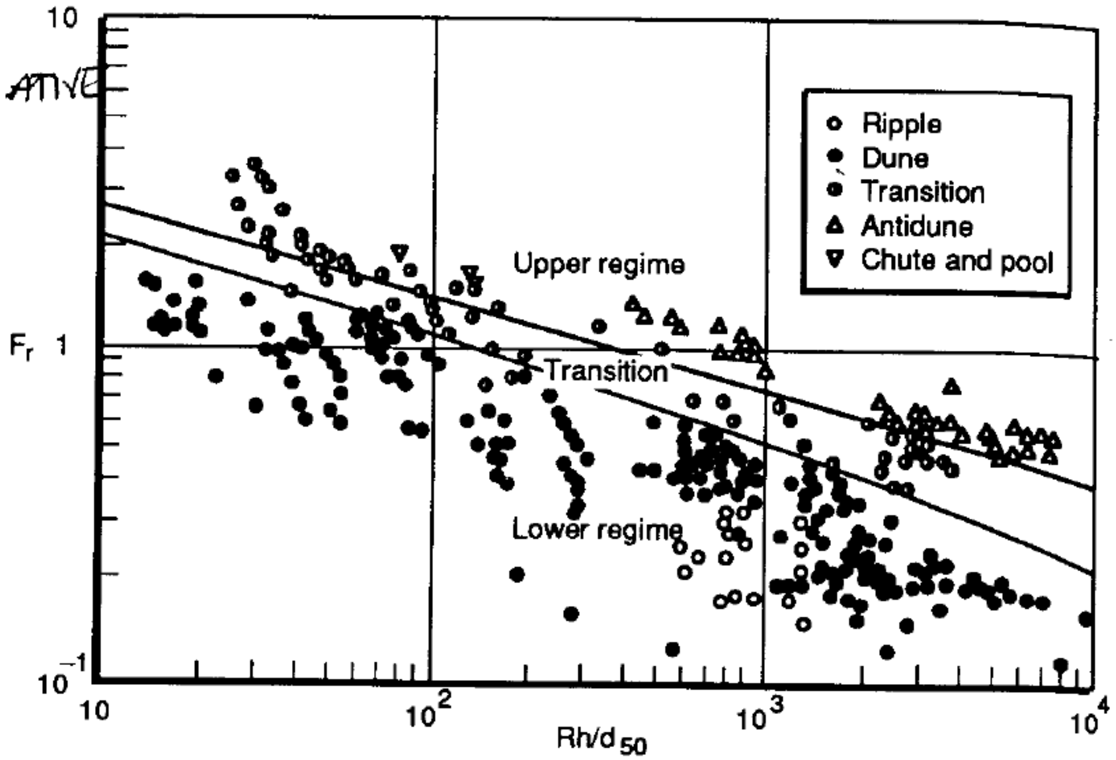


Figure C. 6. Lower and upper regime bedform classification (Athaullah, 1968)

For the lower Cowlitz River the ratio of the radius of curvature to d50 is above 10^3. Froude numbers for the low flows generally fall below 0.3 where **Error! Reference source not found.** indicates lower regime and for higher flows the Froude number is approximately 0.4 and above, indicating upper regime.

From theoretical formulations of the velocity profile, an estimate of the Manning's roughness was made based on the d50 present in the channel. The estimates of Manning's roughness show that as the bedform changes from dunes (lower regime) to upper regime (washed out dunes) the roughness value drops precipitously. A discussion in ASCE (2009) suggests that roughness values dramatically fall as the hydraulics transition bed forms from lower regime/dunes to upper regime. Figure C. 7 shows a figure presented in ASCE (2009) that shows a relationship between Manning's roughness and regime change.

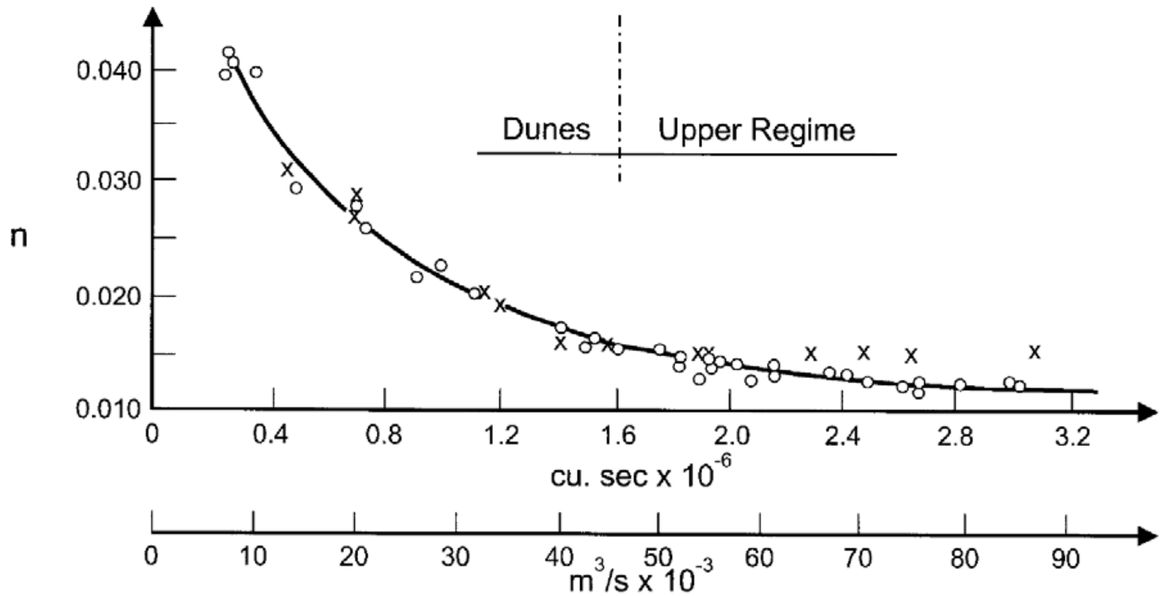


Figure C. 7. Relationship of Manning's Roughness to Regime Change (ASCE 2009)

Below RM 10, for the flood with low AEP where washed out dunes are present, computed roughness values were generally in the range of 0.014 to 0.020. Published tables from Julien (1998), shown in Table C. 8, which relate Manning's coefficient to bedform, supports the contention that near the upper regime conditions where washed out dunes are present; the roughness coefficient can drop to between 0.014 to 0.020.

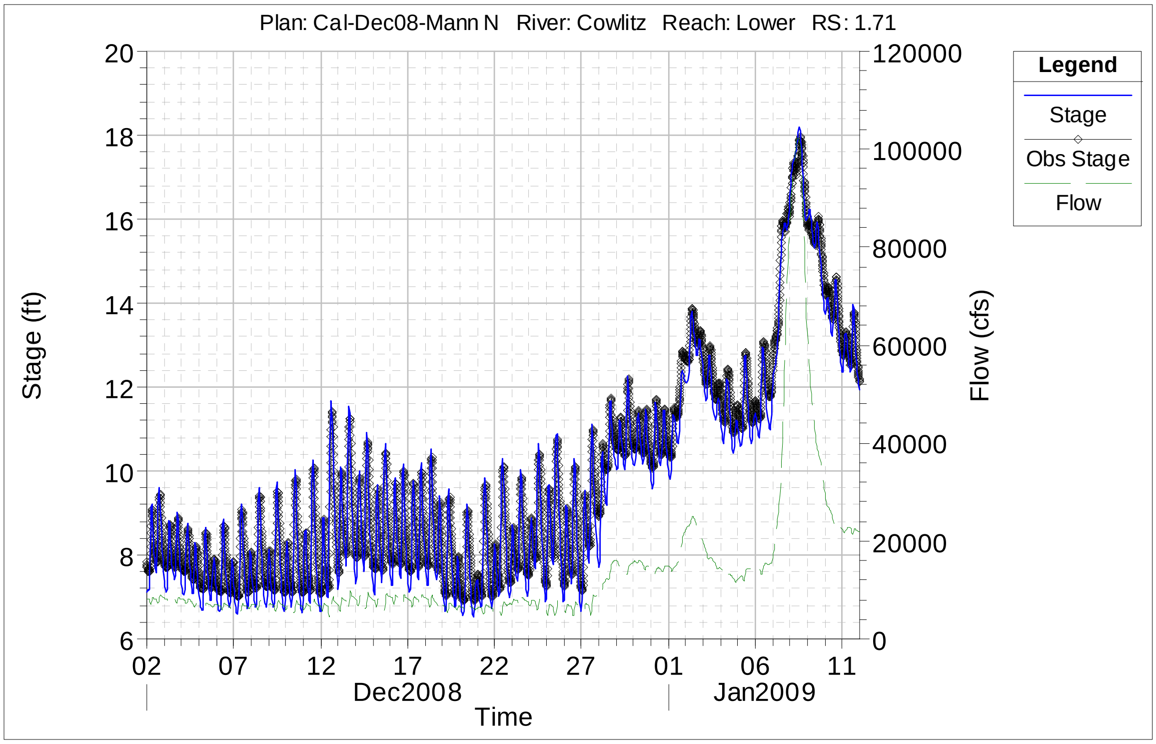
Table C. 8. Typical bedform characteristics (Julien, 1998)

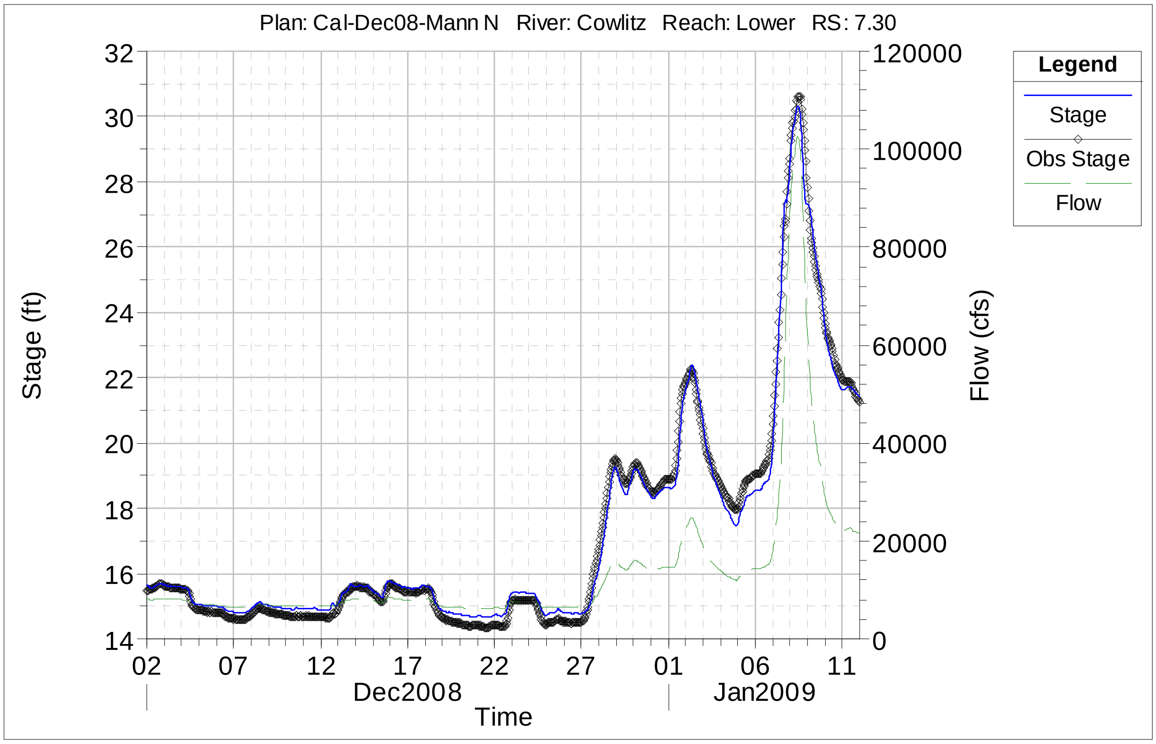
Bedform	Manning coefficient *n*	Concentration (mg/l)	Dominant type of roughness	Bedform surface profiles
Lower flow regime				
Plane bed	0.014	0	Grain	—
Ripples	0.018–0.028	10–200	Form	—
Dunes	0.020–0.040	200–3,000	Form	Out of phase
Washed-out dunes	0.014–0.025	1,000–4,000	Variable	Out of phase
Upper flow regime				
Plane bed	0.010–0.013	2,000–4,000	Grain	—
Antidunes	0.010–0.020	2,000–5,000	Grain	In phase
Chutes and pools	0.018–0.035	5,000–50,000	Variable	In phase

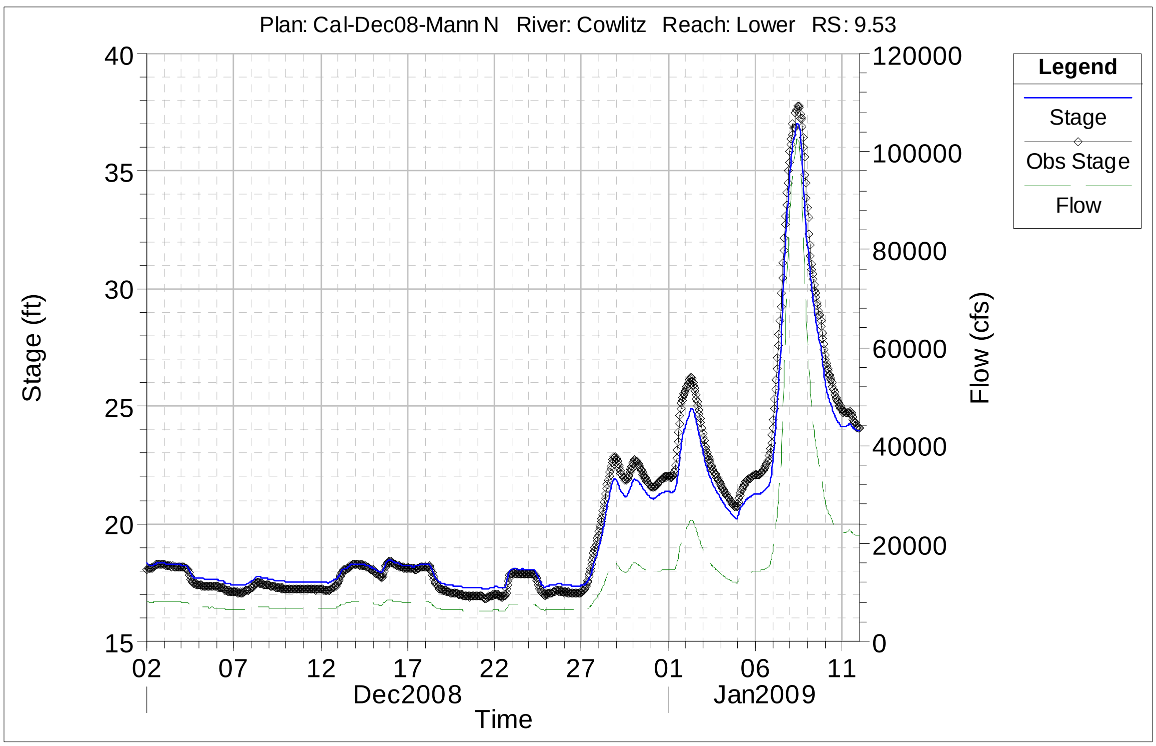
With the range of Manning's roughness determined from research and theoretical computations in mind, two types of calibration were conducted for the lower Cowlitz River. Unsteady calibration to recorded

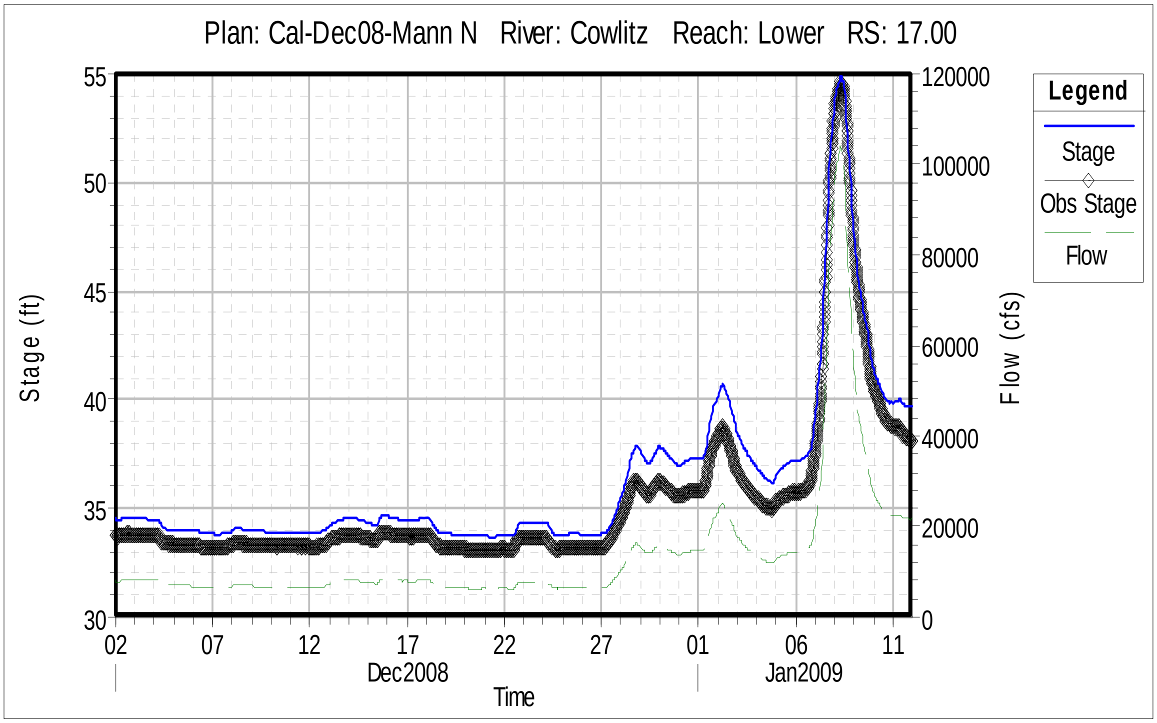
gage data and surveyed high water marks was performed using a Manning's roughness value that varies with respect to discharge and steady flow calibration was also performed using gaged data and surveyed high water marks with a constant Manning's roughness with discharge.

Data from five gages along the Cowlitz are used to calibrate the unsteady flow model for the January 2009 flood event. From RM 0.01 to RM 7.30 Manning's roughness was adjusted with discharge in order to calibrate the stage for the low flows. Roughness values for the peak discharges from RM 0.01 to RM 7.3 are set to 0.02 in the unsteady model. For low flow conditions, leading limb of the flood hydrograph, the roughness values are increased up to a factor of 1.7, or 0.034, in order to match the low flow measured stage hydrograph. Upstream of RM 7.3 to RM 8.07 a roughness value of 0.02 is used, with no variation with flow. Figure C. 8 includes unsteady calibration plots for the five different stage gages.




Plan: Cal-Dec08-Mann N River: Cowlitz Reach: Lower RS: 7.30
Legend
Stage
Obs Stage
Flow
Stage (ft)
Flow (cfs)
32
30
28
26
24
22
20
18
16
14
120000
100000
80000
60000
40000
20000
0
02
07
12
17
22
27
01
06
11
Dec2008
Jan2009
Time


Plan: Cal-Dec08-Mann N River: Cowlitz Reach: Lower RS: 9.53
Legend
Stage
Obs Stage
Flow
Stage (ft)
Flow (cfs)
40
35
30
25
20
15
120000
100000
80000
60000
40000
20000
0
02
07
12
17
22
27
01
06
11
Dec2008
Jan2009
Time



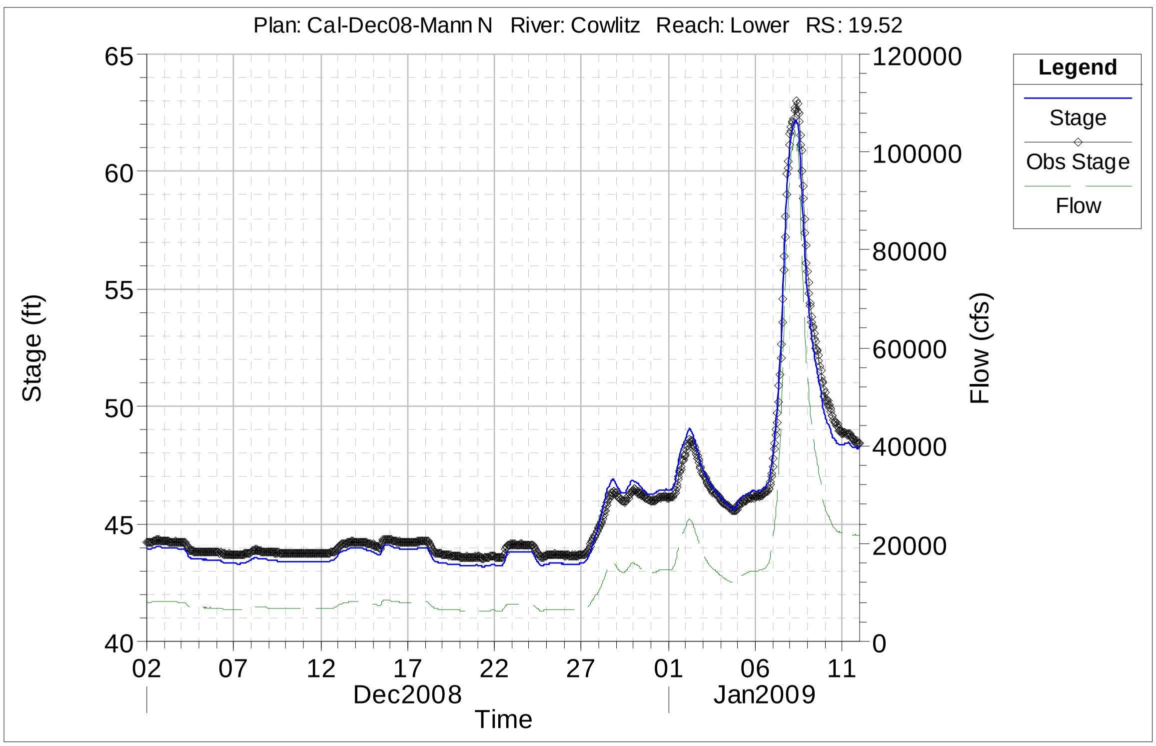


Figure C. 8. Calibration Stage and Flow Hydrographs for the Unsteady Cowlitz River Model

As seen in Figure C. 8, a reasonable calibration was achieved for the unsteady flow modeling. The calibrated roughness values from the unsteady flow model, however, represent a different hydraulic condition than the steady flow model. The unsteady flow model includes, as part of the unsteady computation algorithm, attenuation of the Castle Rock discharge hydrograph. In addition, the unsteady flow model does not include any influence from tributary flow. Since the steady state calibration does

include tributary flow and does not include any attenuation, the calibrated Manning's roughness values from the unsteady flow model is not appropriate for the steady state LOP hydraulic model. Furthermore, since zero probability of failure of the levees exists at low flow, variation of roughness values at low flow will not influence the outcome of the LOP analysis. Table C. 9 summarizes the amount of attenuation realized in the unsteady flow model.

Table C. 9. Attenuation realized in the Unsteady flow model

Model	Cowlitz River Peak Flow (cfs)			
	at Castle Rock	below Arkansas Cr	below Ostrander Cr	below Coweeman River
	2008: RM 20.06	2008: RM 16.1	2008: RM 8.64	2008: RM 1.61
Unsteady Flow Discharge	104,000	104,258	102,490	102,236
Steady Flow	106,000	108,925	110,475	118,975
Difference (Steady – Unsteady)	2,000	4,667	7,985	16,739

Steady state calibration was achieved through the use of eight surveyed high water marks and the peak stage from three gage locations. Since the discharge for steady state conditions include tributary flow and does not include any attenuation from Castle Rock to the Columbia River, lower overall roughness values, especially in the downstream reach, need to be used in order to match the observed data. The calibrated roughness values used in the lower end of the model compare well with the computed roughness values for the washed out dune conditions, and the published data in Table C. 8. Essentially, the roughness values that were used to calibrate the computed water surface profile in the steady state model were verified with gage data, measured high water marks, and computed theoretical roughness values. For this reason model variation from Table 5-2 of EM 1110-2-1619 is based on a *good* estimate of Manning's n reliability.

In the past LOP reports, calibration in the lower portion of the Cowlitz River have been hampered by downstream boundary effect. In 1997 the lower seven miles of the Cowlitz River was calibrated to two high water marks at RM 6.3 and RM 7.2 that were surveyed after a flood event that had a significant backwater influence from the Columbia. While comparisons between the 2009 and 1997 calibration profiles were made, it was recognized that the best available data in 1997 was limited and included some erroneous influences from the Columbia River. While calibration from previous studies have provided a limited basis of comparison for the downstream reach of the Cowlitz River, upstream reaches, where ample calibration data have historically been available, reasonable comparisons can be made with the 2009 calibrated roughness values. Upstream of RM 10, the calibrated Manning's roughness values in 2009 compare reasonably well with previously calibrated values from past LOP reports.

In short calibration to the unsteady gage data represents an upper bound of roughness due to the different conditions present in the steady state conditions. Basic assumptions regarding attenuation and peak flooding renders the roughness coefficients obtained from unsteady calibration invalid for steady state conditions. Therefore, an additional calibration effort was performed to develop calibrated roughness coefficients from steady state conditions and input into FDA. Even though a single peak was considered, the calibration event was a significant flood event and provides a reasonable estimate of roughness values needed to model peak events that could ultimately cause levee failure. The highly dynamic nature of the

reach (2.7 MTons of deposition between 2006 and 2008) make calibration to older event less certain and potentially inappropriate. Roughness values at lower flow events might be perhaps different, however, since the probability of levee failure at these high frequency events is zero the variation in roughness is not essential for the LOP analysis and does not effect the estimate. Low roughness values used in the lower portions of the Cowlitz River were verified with theoretical computations, empirical data, published guidance, and observed water surface. To move away from the observed roughness given the quality of the data and the abundance of theoretical support would not be recommended.

Although Manning's roughness values on the lower end of the river (RM 0.0 to RM 8.0) are low when compared to references such as Chow (1959), the quality of the prediction of observed data provides some validation of the determined roughness coefficients. A review of the hydraulics for the lower 8 miles of the river using Van Rijn's (Julien, 1998) method for predicting bed regime provides evidence of regime changes around the calibration event. Low calibrated Manning's roughness values may, therefore, not be entirely unexpected. In addition, due to the quality of the hydrosurvey in the channel and due to the rich source of observed water surface elevations, the calibrated roughness values, although relatively low, are representative of current channel conditions in the Cowlitz River. Further, apart from future large events that could be used to adjust calibrated roughness values, it is suggested that the Manning's roughness values presented in Table C. 7 should be used in future level of protection studies.

C.5.6. Calibrated Profiles

A water surface profile for the steady state calibration event was computed based on the parameters determined as described in the previous sections. Figure C. 9 shows the resulting computed water surface profile for the January 2009 event compared with the observed flood stages. Manning's roughness values in the channel below RM 6.75 were set to 0.015. Between RM 8.07 and RM 6.76 roughness values were set to 0.019. Although there is still some discrepancy between the observed flood stage and the computed profile for two points at RM 5.76 and 2.69, additional lowering of the Manning's roughness was not thought to be practical. A relatively well calibrated model above RM 8.11 helped support the establishment of the otherwise calibrated computed water surface profile. Results of the calibration computations are provided in Table C. 10.

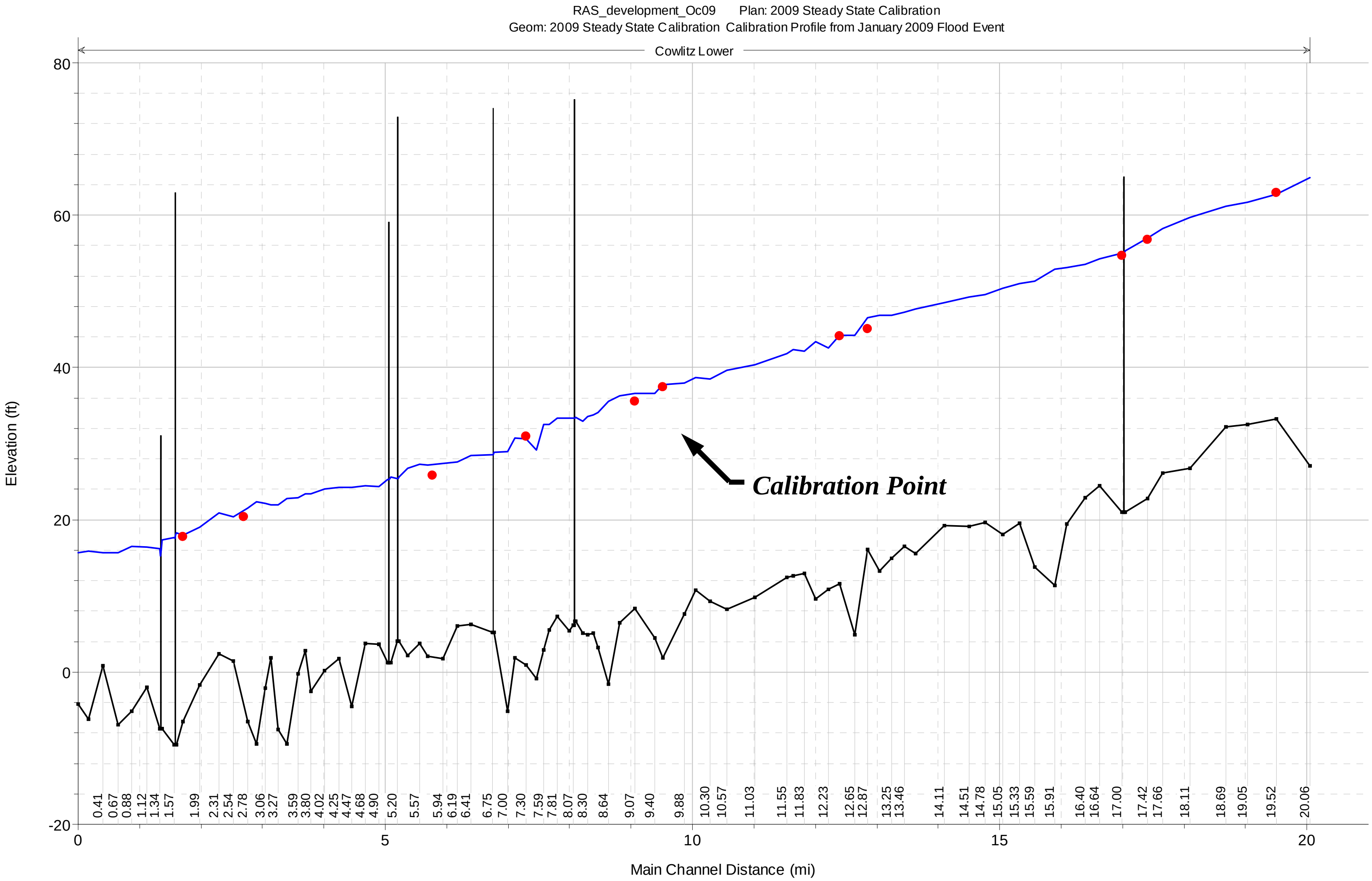


Figure C. 9. Calibration Water Surface Profile from the January 2009 Event

Table C. 10. Results from Calibration Profiles

Observed Point	River Mile	High Water Elevation (ft, NAVD)	Computed WSEL (ft, NAVD)	Difference (ft)
Gerhardt Gardens Gage	1.71	17.76	17.95	0.19
Golf Course	2.78	20.42	21.55	1.13
Upstream of Kelso Bridge	5.94	25.80	27.43	1.63
Ostrander Gage	7.30	30.89	30.63	-0.26
Lexington Park	9.07	35.54	36.59	1.05
Lexington Gage	9.53	37.42	37.72	0.30
Overbank, near Sandy Bend Road	12.40	44.08	44.22	0.14
Horseshoe Bend Point	12.87	45.11	46.53	1.42
Castle Rock Gage	17.00	54.64	54.97	0.33
North end of Castle Rock	17.42	56.74	56.98	0.24
Schmidt Gage	19.52	63.00	62.77	-0.23

C.5.7. Conclusions

Calibration of computed water surface profiles to a series of known observed elevations allow some assurance of the accuracy of the steady state hydraulic model. Calibration results, specifically Manning's roughness values, were used for the 23 water surface profile computations that support the level of protection analysis. Low roughness values obtained from the calibration are supported by transition effects from lower to upper regime. Calibration results indicate some error in the lower end, however, there is limited justification to drive the roughness values lower in this reach. An otherwise reasonably well calibrated model provides assurance of its applicability in the current level of protection analyses.

Energy grade lines for all runs look reasonable and very smooth. The high channel velocities modeled during large events (approx 10 ft/sec) make the model very sensitive to contractions and expansions. Variations in the water surface elevation reflect changes in velocity head primarily due to expansions and contractions in the reach. This may result in raises in water surface in the downstream direction given the large amount of velocity head available.

The calibration effort for the hydraulic model is tailored for the LOP estimate specifically. For the LOP, the focus is on the high flows primarily concerned with the frequency events near failure. Calibration of low flow conditions, or varying roughness values based on discharge, is considered inappropriate for this application.

C.6. UNCERTAINTY

C.6.1. Background

Computed stages from the HEC-RAS model represent mean or expected stages corresponding to a particular discharge and are used to define stage-discharge rating curves for each index location. Uncertainty in the stage-discharge curves is defined using a normal probability density function around computed stages at each index location, with the computed stage representing the median value in the normal distribution. As described in USACE technical guidance (EM 1110-2-1619, 1996) typical sources

of uncertainty include hydraulic model and data limitations and natural variations as represented in gage data. The total uncertainty for these influences on the stage-discharge relation can be estimated as:

$$S_t = (S_{natural}^2 + S_{model}^2)^{0.5} \qquad \textit{Equation 4}$$

Where;

S_t is the standard deviation of the total uncertainty;

$S_{natural}$ is the natural uncertainty; and

S_{model} is the modeling uncertainty.

The supply of sediment from the Toutle watershed resulting from the 1980 Mount St. Helens eruption continues to affect the stage-discharge relationship in the Cowlitz River. To address the uncertainty associated with single event sedimentation, Equation 4 is expanded to include the uncertainty due to sedimentation:

$$S_t = (S_{natural}^2 + S_{model}^2 + S_{sedimentation}^2)^{0.5} \qquad \textit{Equation 5}$$

Where;

S_t is the standard deviation of the total uncertainty;

$S_{natural}$ is the natural uncertainty;

S_{model} is the modeling uncertainty; and

$S_{sedimentation}$ is the uncertainty due to gain or loss of channel capacity due to sedimentation.

In general, the standard deviation of stage uncertainty would be expected to increase with a decrease in data availability, accuracy, and model calibration/validation results. In Equation 5, each component of the total standard deviation represents the summation of the individual random statistics describing each of the major components which are assumed to be normally distributed.

C.6.2. Uncertainty due to Natural Variation ($S_{natural}$)

Uncertainty in the computed stage-discharge curves due to natural variation is developed using sensitivity analysis in the HEC-RAS computer model of the Cowlitz River. Natural variability is represented in the HEC-RAS computer model as variations in Manning's roughness coefficient. Manning's roughness values are established during model calibration and verification processes as described in Appendix C and are tabulated in Table C. 7.

Calibrated Manning's roughness values in the hydraulic model range from 0.015 at the downstream end (RM 0.01) to 0.035 (RM 20.06). By consulting various standard sources, Manning's roughness values for a river condition similar to the Cowlitz River vary by approximately 14 percent above and below average conditions (Chow, 1959; Barnes, 1977; Arcement, and Schneider, 1989). Therefore, where the roughness values determined from the calibration fall within the range given in standard reference material, a 14 percent variation above and below the calibrated roughness values is used to compute the natural variation ($S_{natural}$) in stage.

In the lower portions of the Cowlitz River (below RM 8.07), the calibrated roughness coefficients are generally low compared to the values listed in standard reference material, however, the Manning's roughness values determined from calibration represent verified observed conditions and recorded gage

data. Further, roughness values used in the lower portion of the Cowlitz hydraulic model generally agrees with theoretical estimates of roughness values associated with bedform. Additional uncertainty in Manning's roughness in the lower reaches of the Cowlitz River hydraulic model is included using a greater variation in Manning's roughness from RM 8.07 down to RM 0.01. Table C. 11 summarizes the variation in Manning's roughness values used to compute the natural uncertainty ($S_{natural}$) of the hydraulic model.

Table C. 11: Uncertainty Applied to Manning's Roughness Coefficient

Reach (RM)	Percent Uncertainty	Manning's Roughness Fluctuation		
		Low	Calibrated Value	High
20.06-17.05	14 %	0.031	0.035	0.040
17.00-12.01	14 %	0.028	0.032	0.036
11.83-9.88	14 %	0.025	0.028	0.032
9.53-8.11	14 %	0.019	0.022	0.025
8.07-6.78	30 %	0.015	0.019	0.025
6.75-0.01	30 %	0.012	0.015	0.020

Manning's *n* values are increased and decreased proportionally in the HEC RAS model as part of the sensitivity analysis. Computed stages for the high and low n value parameterization are assumed to be normally distributed and represent the 95% or four standard deviation confidence interval. The difference in computed water surfaces based on the high-low Manning's *n* sensitivity analysis at each index location is considered to bound the computed expected value stage with 95% confidence or four standard deviations. Therefore, the standard deviation representing uncertainty due to natural variation is then computed by Equation 6.

$$S_{natural} = E_{mean}/4 \qquad \textit{Equation 6}$$

Where;

$S_{natural} =$ natural uncertainty in feet

$E_{mean} =$ mean or expected value stage difference between upper and lower water surface profiles developed using the high and low n value estimates for Manning's *n*.

Natural uncertainty values, computed from the Manning's roughness sensitivity analysis, are compiled for each index point and each exceedance probability discharge in **Error! Reference source not found.**.

C.6.3. Uncertainty due to Model and Data Limitations (S_{model})

Criterion from Table 5-2 in EM1110-2-1619 is used to develop the uncertainty values with respect to the computer model; this table is reproduced below (Table C. 12). Due to the quality of the calibration data (gage data, surveyed high water marks, and comparisons with theoretical roughness values), the Manning's value reliability of *Good* is used with *Cross Section Based on Field Survey.* As a result, the model uncertainty is set uniformly to 0.3 ft.

Table C. 12. Minimum standard deviation of error in stage (Table 5-2 in EM 1110-2-1619)

Table 5-2
Minimum Standard Deviation of Error in Stage

	Standard Deviation (in feet)	
Manning's n Value Reliability[1]	Cross Section Based on Field Survey or Aerial Spot Elevation	Cross Section Based on Topographic Map with 2-5' Contours
Good	0.3	0.6
Fair	0.7	0.9
Poor	1.3	1.5

[1] Where good reliability of Manning's *n* value equates to excellent to very good model adjustment/validation to a stream gauge, a set of high water marks in the project effective size range, and other data. Fair reliability relates to fair to good model adjustment/ validation for which some, but limited, high-water mark data are available. Poor reliability equates to poor model adjustment/validation or essentially no data for model adjustment/validation.

C.6.4. Uncertainty Due to Sedimentation ($S_{sedimentation}$)

Sedimentation uncertainty is a difficult parameter the quantitatively assess. The approach used in this study investigates multiple methods to ascertain changes in water surface profiles due to sedimentation variation. Compilation of various approaches was used as guidance in determining the uncertainty due to sedimentation. Final values for the sedimentation uncertainty are included in **Error! Reference source not found.**.

C.6.4.1.Bed Form Analysis

Changing of bed form regime can have an effect on flow profiles during high water events. If sufficient stream power exists, regime change in the bed form can result in reduction of bed form roughness and a resulting decrease in channel roughness. This relationship is shown in Figure C. 10. It is often found that the discharge at which the dunes are obliterated is a little below bank-full in sand bed streams with medium to high bed slopes (ASCE, 2009; pg. 99).

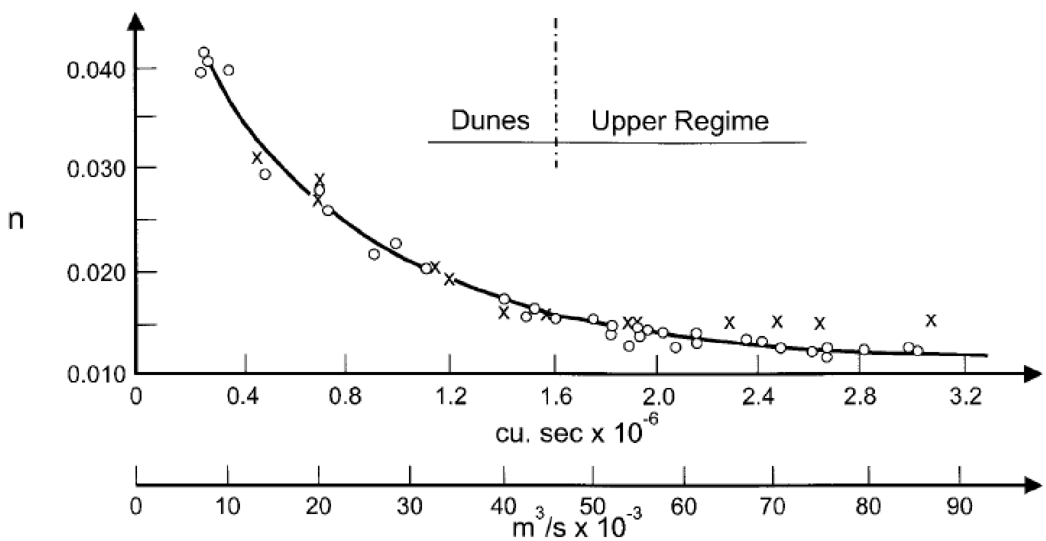


Fig. 2-47. Variation of Manning's n with flow discharge for the Padma River, Bangladesh. (o) observations of Stevens and Simmons and (x) computations of Chollet and Cunge (1980).

Figure C. 10. Effects of Regime change on Manning's Roughness (Vanoni, 2008)

Van Rijn's bedform prediction method was utilized to determine the regime in the Cowlitz below Castle Rock, WA (Julien, 1998). The results from the regime prediction analysis, shown in Figure C. 11,

indicate that for the range of flows considered in the level of protection analysis a wide range of bedforms can be expected. Figure C. 11 shows that at high discharges, upper regime is likely, however, the hydraulics of the Cowlitz River spans a wide range of regime conditions.

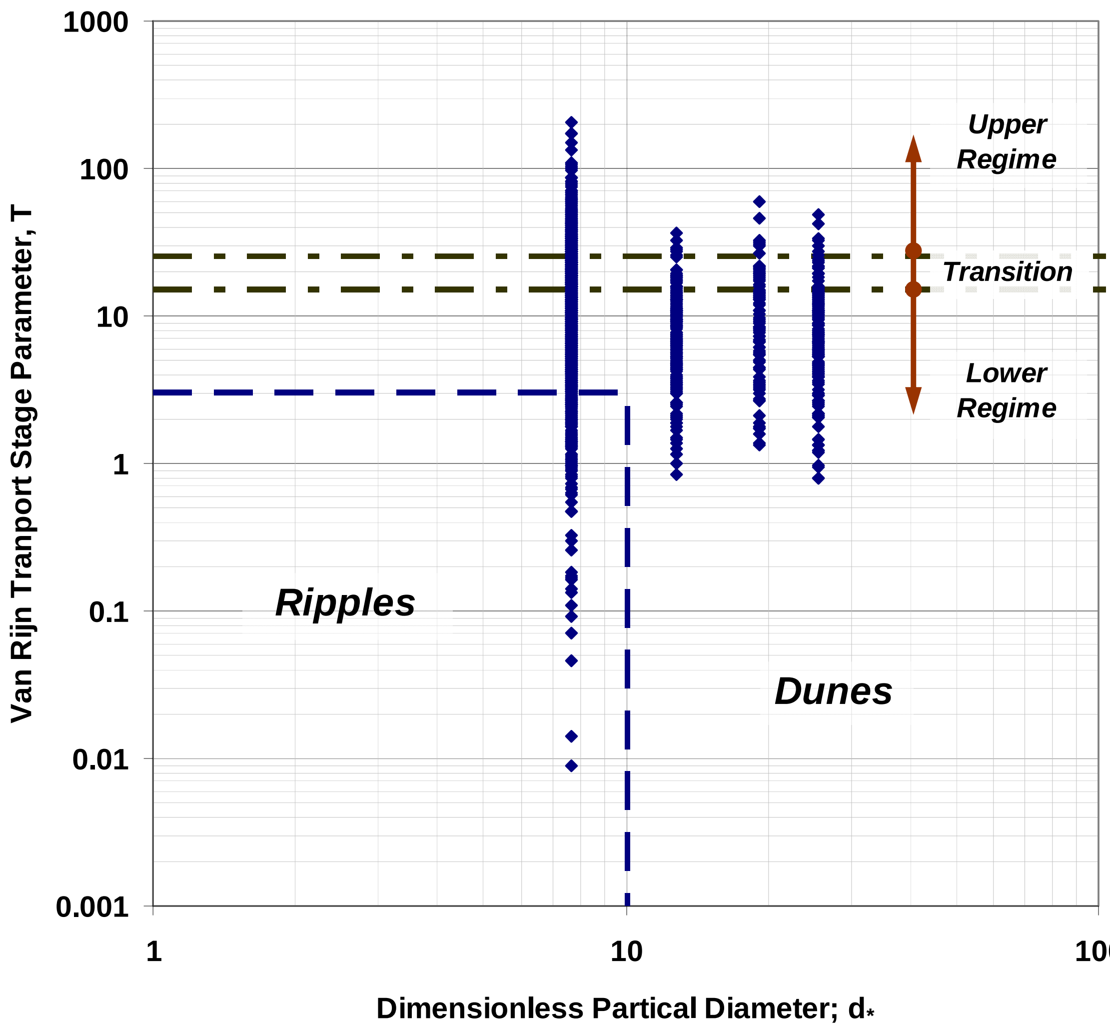


Figure C. 11. Regime Determination for the Cowlitz River below Castle Rock, WA

From the regime predictions, the Manning's roughness values can be computed from theoretical principles of velocity profiles. Manning's roughness values are computed for the entire range of Transport Stage Parameters shown in Figure C. 11. At each cross section the range of computed Manning's Roughness values can be determined and then applied to the calibrated Manning's roughness for the corresponding cross section. The range in computed Manning's roughness values represents the changes in model calibration parameters due to the migration and evolution of bedforms through the system. As a result, the stage computed from applying the range of Manning's roughness to the hydraulic model represents, to some degree, the uncertainty in the bed conditions during a flood event. It is assumed that the variation in stage caused by the dune formation, computed in the manner described above, represents 2 standard deviations on either side of the mean. Figure C. 12 shows a plot of the resulting variation in computed stage based on the changes in the bed form conditions. As illustrated in Figure C. 12, the stage variation is lower for extremely high discharges. This result supports the notion that at the high discharges, where upper regime persists, the bed will plane out and the roughness will be

lower than at lower discharges. For the purposes of the level of protection analysis, Figure C. 12 suggests that changes in bed form for discharge relavant to the LOP analysis (i.e. approximately the 100-year frequency event) can cause sedimentation uncertainty to vary from 0.6 ft to 1.2 ft.

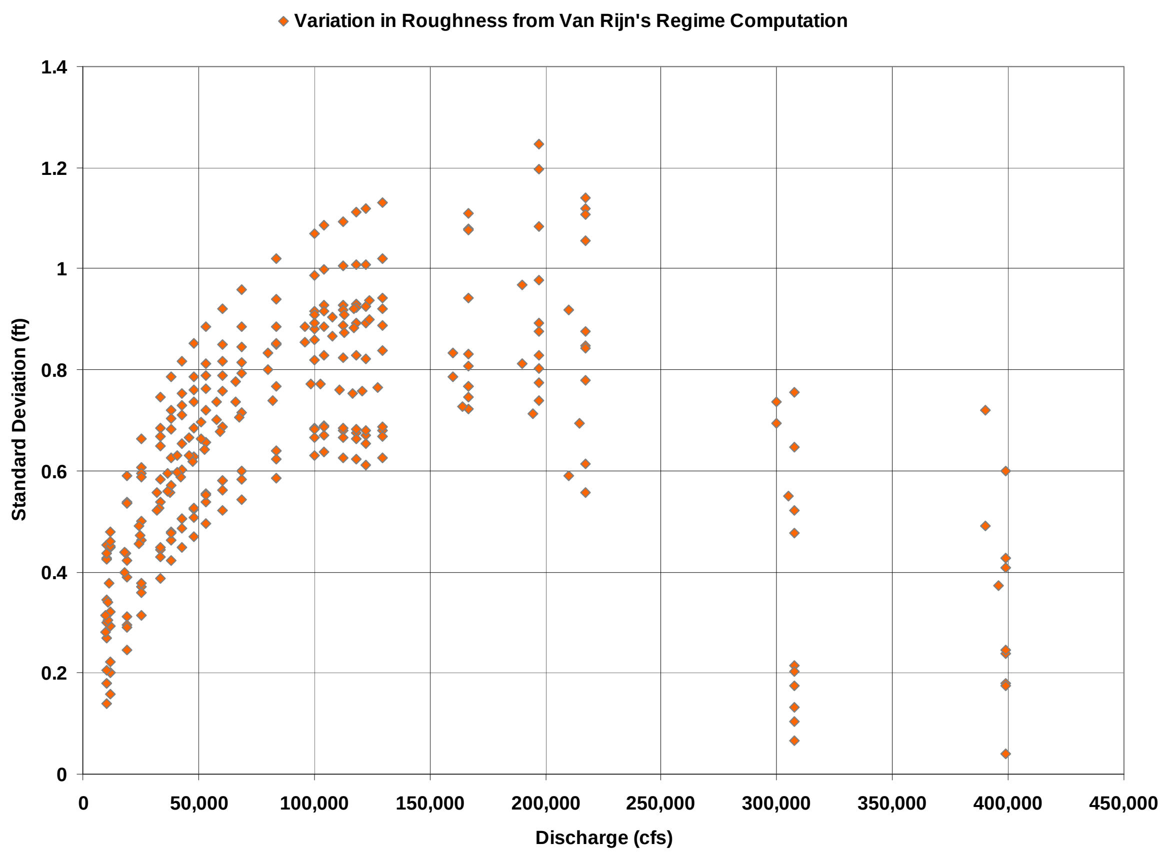


Figure C. 12. Variation in Stage based on changes in Bed Form Conditions

C.6.4.2. Variation in Bed Elevation at USGS Castle Rock Gaging Station (14243000).

The USGS regularly survey a cross section on the upstream side of “A” street bridge over the Cowlitz River adjacent to the Castle Rock gaging station. Frequent bed elevation and velocity surveys are required to maintain the gage rating curve due to rapid changes in the system. This source of data is the only one available on the Lower Cowlitz that measures changes in sedimentation on a near monthly basis with additional survey during flood event peaks. To this end, it allows for an observed range of bed elevation change to be determined and allows a relationship between bed change and peak discharges to be ascertained.

Each cross section was reduced to a single average bed elevation from bank-toe to bank-toe. These elevations along with observed flow at Castle Rock are plotted in Figure C. 13. Annual variation in bed elevation ranges from 1.0 to 1.5 ft with the highest flow year showing 4.0 ft variation at this location. The data persistently shows that scour rapidly occurs during larger event but the bed quickly recovers to pre-event average conditions during the recession limb of the hydrograph. The exception to this is the increase in average bed elevation in the months following the November 2006 flood event when the average bed elevation rose 1 ft with no clear signs of recovery. Both scour and deposition can occur during a given year. Rapid changes in bed elevation due to high flows tend to scour at this location.

Without additional data it is uncertain whether the observed scour during high flow is a local or a reach phenomena. The location near the bridge is a minor constriction during bankfull or smaller events. During the January 2009 event, overbank flow upstream and downstream of the bridge was observed indicating that the bridge opening becomes a more significant constriction with a potential localized velocity increase near the bridge. This could result in a localized scour, however natural and man-made constrictions and hardpoints persist throughout the Lower Cowlitz reach potentially extending this logic to multiple locations. This indicates that any bias toward a positively skewed distribution of sediment uncertainty should be tempered by the trend toward scour at peak flows observed at Castle Rock.

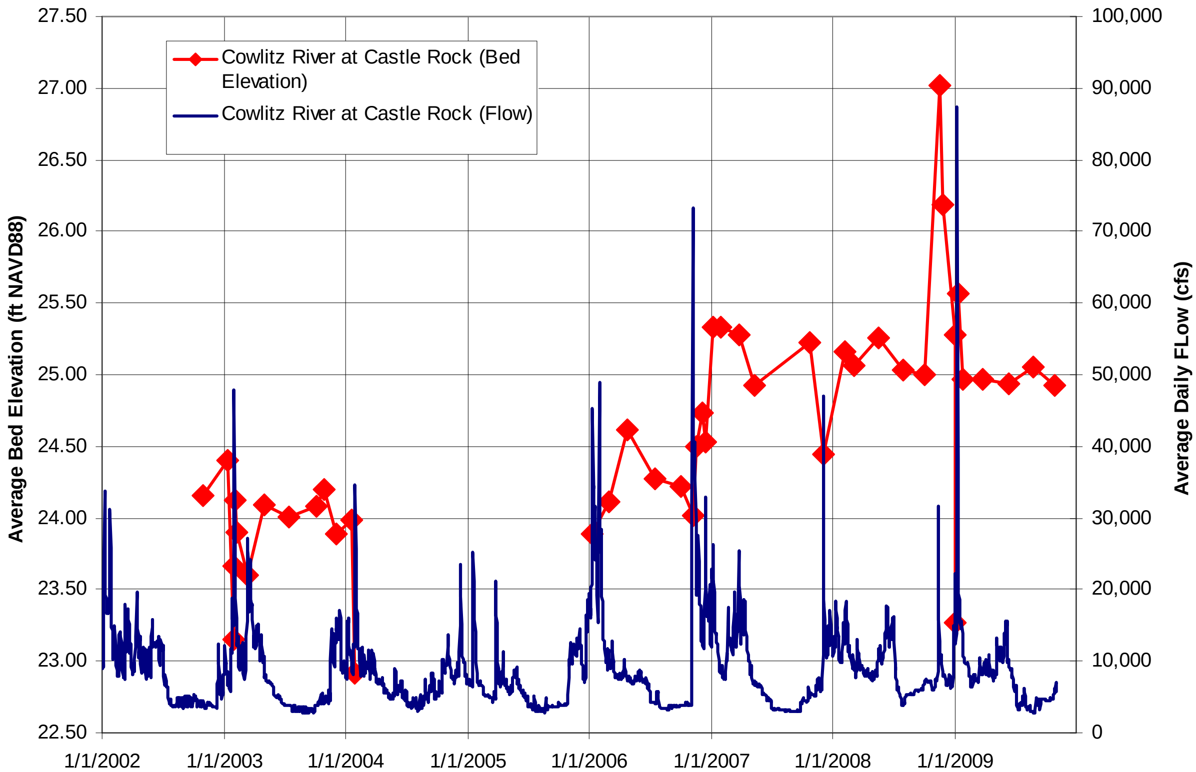


Figure C. 13. Changes in Bed Elevation for Cross Sections Surveyed at the Caste Rock Bridge

C.6.4.3. Gage Analysis

Along the Cowlitz River, in the reach from the confluence with the Toutle River to the Columbia River, data from two gage locations are used to provide some insight into the variability or uncertainty in water surface elevation at the levee index points. A continuous record of discharge and stage data has been recorded at Castle Rock from a gage operated and maintained by the USGS (Gage Number 14243000). Data from 1990 to the present was used to compare the relationship between recorded stage and the reported discharge. Additionally, stage records from 1990 to the present were obtained from a gage at Kelso, Washington which is operated and maintained by NOAA's Northwest River Forecast Center (Gage KELW1). Information regarding the analysis of this data is described in the following two sections.

C.6.4.3.1. Castle Rock Gage

For the purposes of determining variation in stage, data from 1990 to the present was obtained from the USGS Castle Rock gage. Stage data from this gage is reported by the USGS to be in NAVD 88. Plotting discharge versus stage for this time period allowed estimation of the uncertainty in stage for a given

discharge. This analysis effectively examines the range of shifts in the gage rating curve over time throughout the range of observed discharges at the Castle Rock Gage. Figure C. 14 shows estimates of stage uncertainty for given discharges from data obtained from the Castle Rock gage.

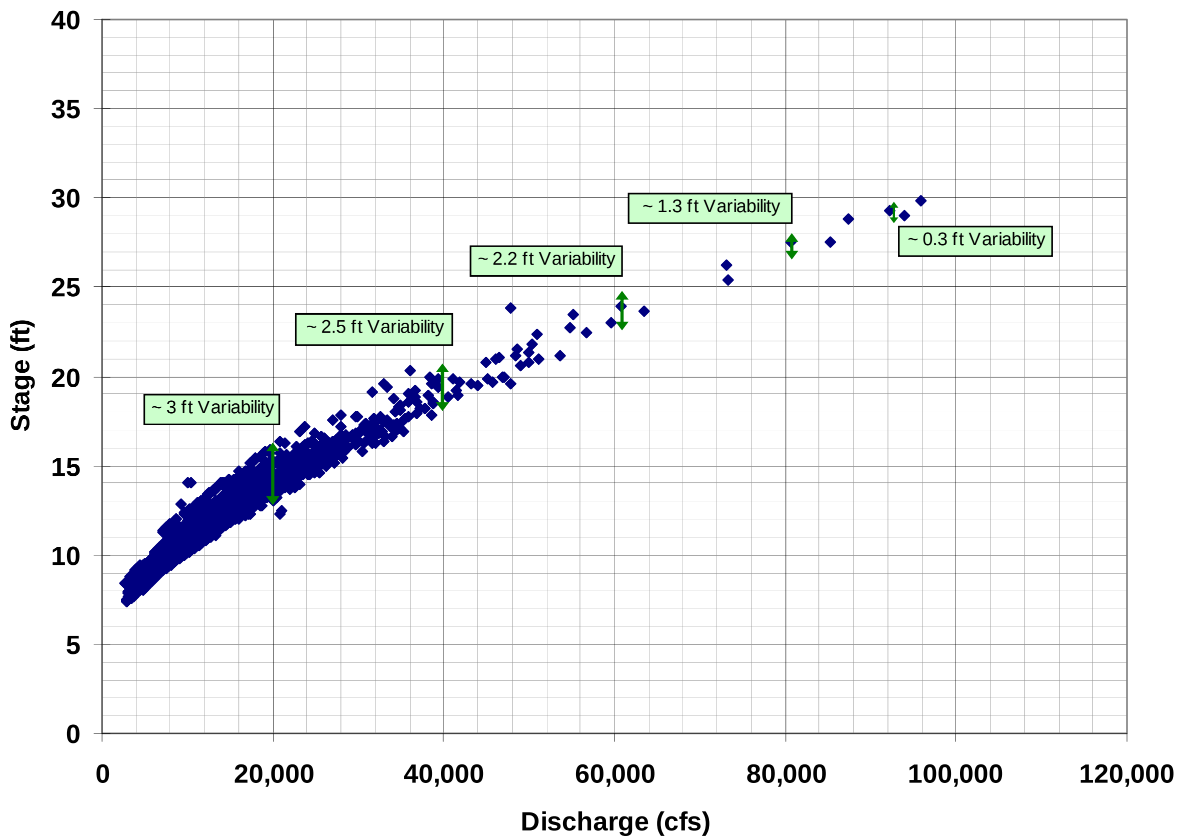


Figure C. 14. Discharge vs. Stage from 1990 to present at Castle Rock

Stage variability, as shown in Figure C. 14 ranges from 3 feet to 0.3 feet and tends to decrease for increasing discharge. Variations in stage measured from gage data incorporate changes in cross section area at the gage location due to sedimentation and/or obstructions. It is estimated that the variability estimate provided in Figure C. 14 represent 95 percent of the possible variation in stage, or two standard deviations. As such, the variance, *S*, for a single standard deviation is computed as 1/4th the estimated variability shown in Figure C. 14. Table C. 13 summarizes the computed variance at the Castle Rock gage for given discharges.

Table C. 13. Estimated variance from gage data at Castle Rock, WA.

Discharge (cfs)	Estimated Variability in Stage (ft)	Variance, *S* (ft)
20,000 cfs	3 ft	0.75 ft
40,000 cfs	2.5 ft	0.63 ft

60,000 cfs	2.2 ft	0.55 ft
80,000 cfs	1.3 ft	0.33 ft
92,000 cfs	0.3 ft	0.075 ft

Variance estimates presented in Table C. 13 are computed for the range of discharges available and are meant to provide a representative range of variability for the stages in the upper portions of the Cowlitz River near Castle Rock.

C.6.4.3.2. Kelso Gage

Stage data from the Kelso gage was obtained from 1990 to the present. Variations in stage are computed by comparing the stage data with discharges from the Castle Rock gage. From conversations with the NOAA NWRFC representative, internal knowledge of gage maintenance, and a review of the stage data itself, several periods of questionable data are identified and ultimately discarded in this comparison. The final comparison of stage to discharge for the Kelso gage included a dataset from January 1, 1998 to November 1, 1999; May 1, 2000 to January 1, 2006 and October 1, 2009 to present as the period of reliable data. Figure C. 15 shows the resulting plot of stage versus discharge for this time period with variation in stage identified and estimated.

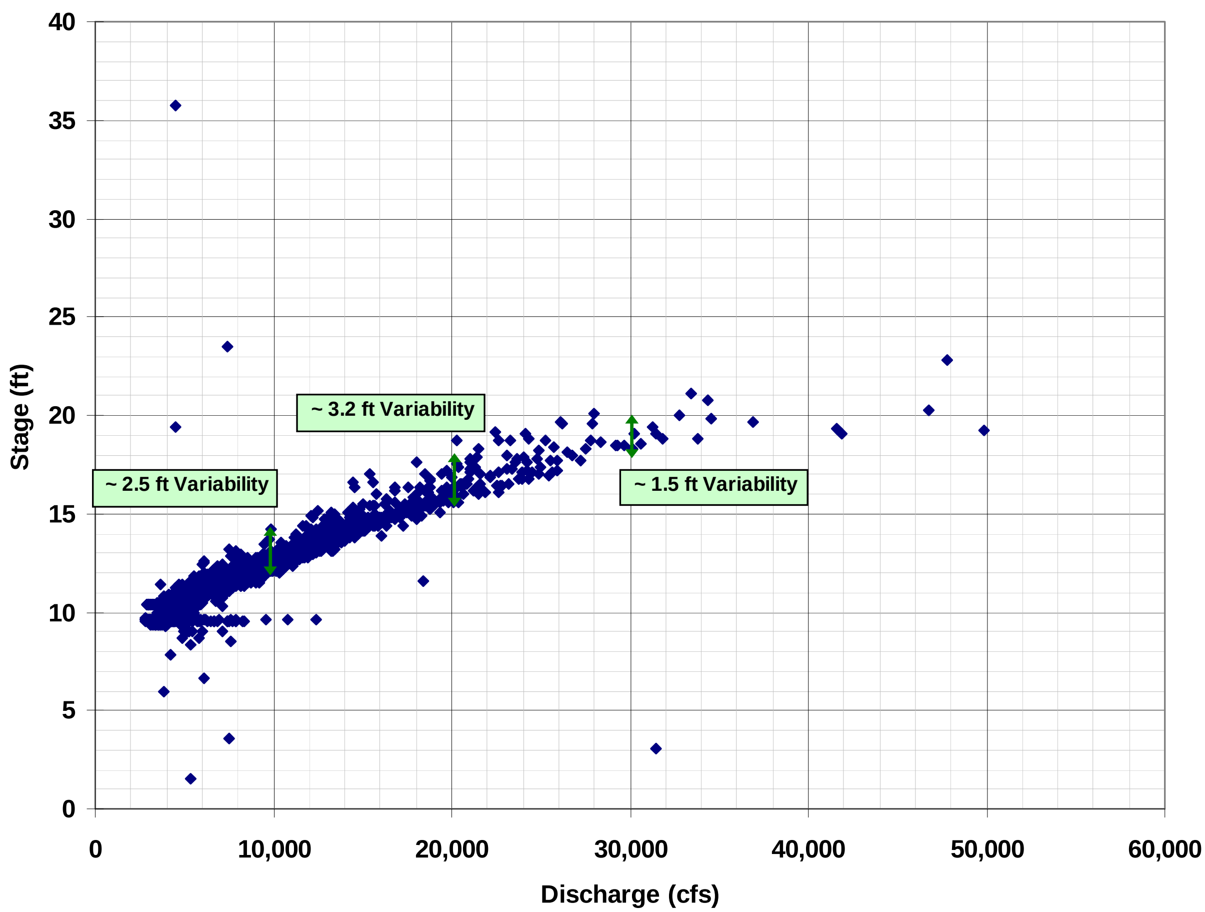


Figure C. 15. Discharge vs. Stage for the Kelso Gage

Variability in stage, as estimated from Figure C. 15 ranges from 3.2 ft to 2.5 ft and is expected to incorporate 95 percent of possible data. While the estimated variability is not strictly decreasing with increasing discharge, a generally decreasing trend can be noted. A higher variability of 3.2 ft at 20,000 cfs is likely due more to scatter in gage readings than actual variability caused by changes in cross sectional area or blockages. Estimation of the variation for one standard deviation is computed in a similar manner as Castle Rock data as 1/4th of the estimated values in Figure C. 15.

Data from the Kelso gage differs from the data from the Castle Rock gage in that the Cowlitz River at Kelso is influenced, to some degree, by tidal phenomena from the Columbia. Estimation of tidal effects at the Kelso gage are made using results from an unsteady one-dimensional hydraulic model of the Lower Cowlitz that had been calibrated to gage data and high water observations. On average, the tidal fluctuation are measured to be 0.28 ft which is subtracted from measured variability in Figure C. 15. Final estimates of stage variability at the Kelso gage are summarized in Table C. 14.

Table C. 14. Estimated variance from gage data at Kelso, WA

Discharge	Estimated Variability in Stage	Tidal Fluctuations	Adjusted Variability in Stage	Variance, *S*
(cfs)	(ft)	(ft)	(ft)	(ft)
10,000 cfs	2.5 ft	0.28	2.22	0.56 ft
20,000 cfs	3.2 ft	0.28	2.92	0.73 ft
30,000 cfs	1.5 ft	0.28	1.22	0.31 ft

C.6.4.4. Observed Deposition

Since 1990, eight difference sets of bathymetry and cross section survey data has been collected, as summarized in Table C. 15. Hydraulic models were developed for these various bathymetry surveys to assess changes in hydraulic conditions along the Cowlitz River.

Table C. 15. Dates of Available Bathymetry data Collected on the Cowlitz River

Date	Number of Cross Sections Surveyed
May 1990	92
August 1991	68
July 1992	68
Summer 1996	80
Summer 2003	64
April 2006	67
December 2006	72
June 2008	88

Comparisons between the various surveys are made in order to assess the degree of aggradation and degradation along the Cowlitz River. Volume is computed in terms of tons per mile along the Cowlitz River and was then annualized to represent a yearly volume of sediment. Maximum and minimum volumes are determined from computed amount of sediment per reach per year (tons/mi/year) computed from the cross section surveys. For the purposes of uncertainty analysis, only the years after the SRS was filled are used to compute the maximum and minimum values of sediment moving through the Cowlitz River. A plot of volume estimates from the cross section survey data is shown in Figure C. 16.

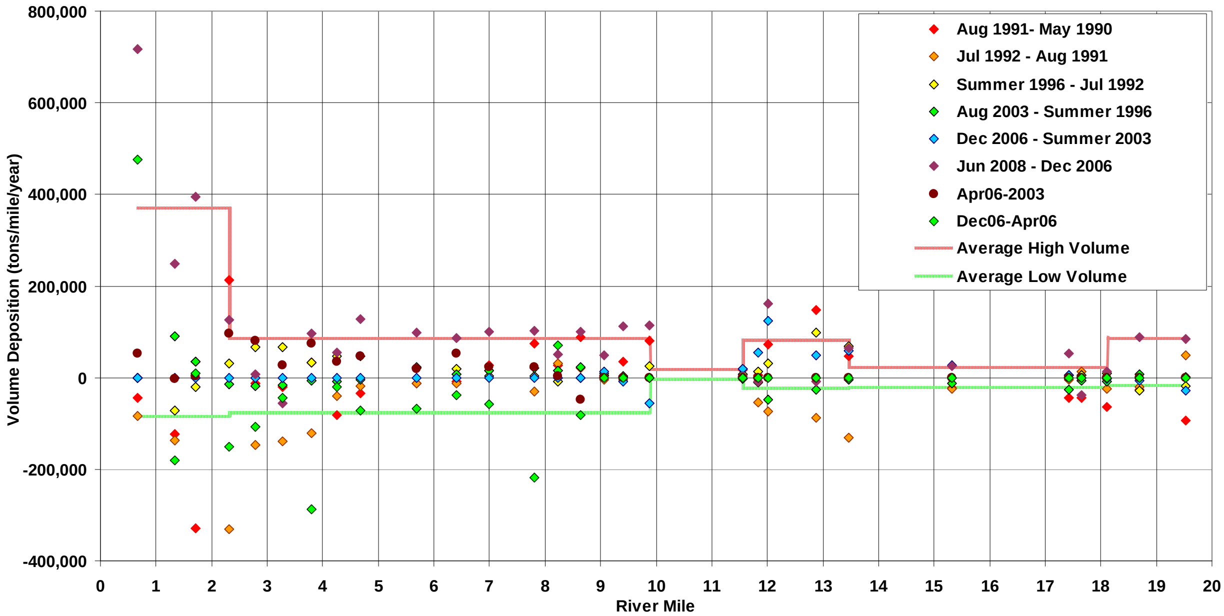


Figure C. 16. Volume of Sediment Computed for the Cowlitz using Surveyed Cross Section Data

In order to assess the temporal variability in the amount of measured sediment volume, the Cowlitz River is divided into six representative reaches. Within each reach an average of the maximum and minimum volume estimates is computed. Average maximum and minimum values are assigned to each reach as shown by the dashed lines in Figure C. 16. Average volumes assigned to each reach from the maximum and minimum values are considered to represent 95 percent of the population. Thus, the average maximum and minimum values are considered to include two standard deviations about the mean.

Assigned maximum and minimum volumes are used to compute the amount of aggradation and degradation in feet that might occur between the channel banks for each cross section along the Cowlitz River. Cross sections are then adjusted based on this estimate of aggradation and degradation for maximum and minimum sediment volumes. Two steady HEC-RAS models are compiled for the condition of high sediment volumes and for low sediment volumes, respectively. Computed water surface profiles from the two HEC-RAS models are used to represent the stage variability due to the uncertainty in sedimentation. Although the sediment uncertainty for the level of protection analysis is not predictive in nature, using historic data allows some sense of the Cowlitz River's historical variation in sediment load. Considering that on any given year the sediment might fluctuate from the estimated maximum to the estimate minimum values the resulting computed water surface profile is thought to represent only 66 percent of the variance or a total of two standard deviations.

C.6.4.5. Conclusions of Sedimentation Uncertainty

Compared to natural and model uncertainty, determination of the sediment uncertainty is particularly difficult, confounded by the relative lack of guidance. Because no direct solution of the sedimentation uncertainty is available, an effort is made to look at various aspects of the sediment uncertainty and combine them to get an overall picture of what the uncertainty might be. Each method used to evaluate the sedimentation uncertainty possesses inherent strengths and weaknesses which thwarted the development of any trend in the computed data. In the end, determination of the sediment uncertainty is based as much on engineering judgment as on deterministic methodology. Figure C. 17 shows a plot of each method considered in the determination of the sediment uncertainty. From a critical evaluation of each method the final sediment uncertainty is chosen to be 0.25 ft for the Castle Rock Levees (RM 15.91 and above) to 0.70 ft below Castle Rock normally distributed about the mean.

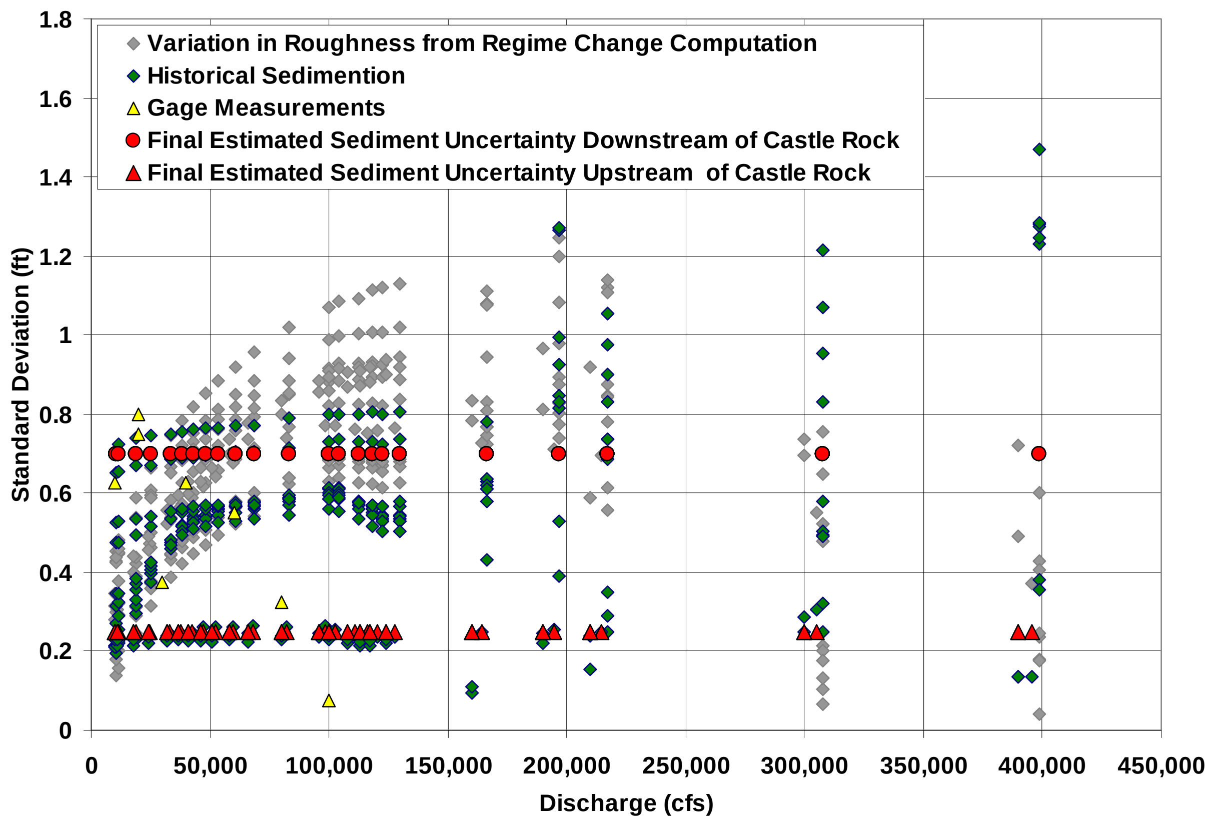


Figure C. 17. Sedimentation Variance based on a Variety of Methodologies

C.7. UNCERTAINTY RESULTS

Uncertainty in the stage-discharge curves developed for the 2009 LOP estimates were based on the combination of the natural, model, and sedimentation uncertainty as described above. Computationally, the three components of the stage discharge uncertainty are combined based on Equation 7. Table C. 16 represents the individual components of uncertainty by type for each exceedance probability event.

$S_t = (S_{natural}^2 + S_{model}^2 + S_{sedimentation}^2)^{0.5}$ *Equation 7*

Where;

S_t is the standard deviation of the total uncertainty;

$S_{natural}$ is the standard deviation of the natural uncertainty

(obtained by considering a range of Manning's roughness);

S_{model} is the standard deviation of the model uncertainty

(obtained from Table 5-2 in EM 1110-2-1619);

$S_{sedimentation}$ is the standard deviation of the sedimentation uncertainty

(obtained from considering changes in sedimentation over a 10 year history, variation in bedform during storm events, variation of recorded gage information, and engineering judgment; see Section 4.3).

Table C. 16. Uncertainty Results for Each Index Point

	Rating Curve from Calibrated Hydraulic Model				Uncertainty in Rating Curve			
					Natural	Sedimentation	Model	Total
	RM	Frequency	Discharge	Computed Water Surface	Sn	Ssed	Smodel	Stotal
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft)
Longview 4 (LVIP 4)	3.27	0.9999	10400	11.33	0.1275	0.7	0.3	0.772
	3.27	0.99	11700	11.65	0.1425	0.7	0.3	0.775
	3.27	0.95	19000	13.01	0.22	0.7	0.3	0.793
	3.27	0.9	25200	13.91	0.2825	0.7	0.3	0.812
	3.27	0.8	33400	15.05	0.3475	0.7	0.3	0.837
	3.27	0.7	38100	15.66	0.3775	0.7	0.3	0.850
	3.27	0.6	42800	16.3	0.3975	0.7	0.3	0.859
	3.27	0.5	48000	17	0.415	0.7	0.3	0.867
	3.27	0.4	53200	17.63	0.44	0.7	0.3	0.880
	3.27	0.3	60500	18.47	0.465	0.7	0.3	0.892
	3.27	0.2	68800	19.38	0.4825	0.7	0.3	0.902
	3.27	0.1	83400	20.82	0.5225	0.7	0.3	0.924
	3.27	0.05	99900	22.25	0.56	0.7	0.3	0.945
	3.27	0.04	104100	22.63	0.5675	0.7	0.3	0.950
	3.27	0.02	112600	23.58	0.56	0.7	0.3	0.945
	3.27	0.01	118200	24.16	0.5575	0.7	0.3	0.944
	3.27	0.007	122400	24.64	0.55	0.7	0.3	0.939
	3.27	0.005	129700	25.22	0.5625	0.7	0.3	0.947
	3.27	0.002	166500	27.4	0.6525	0.7	0.3	1.003
	3.27	0.001	197000	29.04	0.7075	0.7	0.3	1.039
	3.27	0.0008	217100	30.02	0.9475	0.7	0.3	1.216
	3.27	0.0005	307500	35.59	0.0775	0.7	0.3	0.766
	3.27	0.0001	398800	39.86	0.3925	0.7	0.3	0.857

	Rating Curve from Calibrated Hydraulic Model				Uncertainty in Rating Curve			
					Natural	Sedimentation	Model	Total
	RM	Frequency	Discharge	Computed Water Surface	Sn	Ssed	Smodel	Stotal
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft)
Longview 3 (LVIP 3)	3.59	0.9999	10400	11.44	0.1575	0.7	0.3	0.778
	3.59	0.99	11700	11.77	0.175	0.7	0.3	0.781
	3.59	0.95	19000	13.21	0.2575	0.7	0.3	0.804
	3.59	0.9	25200	14.19	0.32	0.7	0.3	0.826
	3.59	0.8	33400	15.39	0.385	0.7	0.3	0.853
	3.59	0.7	38100	16.04	0.4125	0.7	0.3	0.866
	3.59	0.6	42800	16.71	0.435	0.7	0.3	0.877
	3.59	0.5	48000	17.43	0.455	0.7	0.3	0.887
	3.59	0.4	53200	18.09	0.4775	0.7	0.3	0.899
	3.59	0.3	60500	18.97	0.5025	0.7	0.3	0.912
	3.59	0.2	68800	19.93	0.52	0.7	0.3	0.922
	3.59	0.1	83400	21.43	0.555	0.7	0.3	0.942
	3.59	0.05	99900	22.95	0.5925	0.7	0.3	0.965
	3.59	0.04	104100	23.35	0.6	0.7	0.3	0.970
	3.59	0.02	112600	24.3	0.595	0.7	0.3	0.966
	3.59	0.01	118200	24.89	0.5925	0.7	0.3	0.965
	3.59	0.007	122400	25.37	0.5875	0.7	0.3	0.962
	3.59	0.005	129700	25.98	0.595	0.7	0.3	0.966
	3.59	0.002	166500	28.37	0.68	0.7	0.3	1.021
	3.59	0.001	197000	30.2	1.1125	0.7	0.3	1.348
	3.59	0.0008	217100	33.35	1.0375	0.7	0.3	1.287
	3.59	0.0005	307500	36.96	0.145	0.7	0.3	0.775
	3.59	0.0001	398800	41.21	0.4025	0.7	0.3	0.861
Kelso 4 (KLIP 4)	3.7	0.9999	10400	11.57	0.16	0.7	0.3	0.778
	3.7	0.99	11700	11.91	0.1775	0.7	0.3	0.782
	3.7	0.95	19000	13.4	0.26	0.7	0.3	0.805
	3.7	0.9	25200	14.41	0.3175	0.7	0.3	0.825
	3.7	0.8	33400	15.65	0.3775	0.7	0.3	0.850
	3.7	0.7	38100	16.32	0.405	0.7	0.3	0.863
	3.7	0.6	42800	17	0.4275	0.7	0.3	0.873
	3.7	0.5	48000	17.74	0.445	0.7	0.3	0.882
	3.7	0.4	53200	18.41	0.47	0.7	0.3	0.895
	3.7	0.3	60500	19.31	0.4925	0.7	0.3	0.907
	3.7	0.2	68800	20.29	0.5125	0.7	0.3	0.918
	3.7	0.1	83400	21.84	0.5475	0.7	0.3	0.938
	3.7	0.05	99900	23.42	0.5825	0.7	0.3	0.959
	3.7	0.04	104100	23.83	0.5875	0.7	0.3	0.962

	Rating Curve from Calibrated Hydraulic Model				Uncertainty in Rating Curve			
					Natural	Sedimentation	Model	Total
	RM	Frequency	Discharge	Computed Water Surface	Sn	Ssed	Smodel	Stotal
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft)
	3.7	0.02	112600	24.79	0.5825	0.7	0.3	0.959
	3.7	0.01	118200	25.39	0.58	0.7	0.3	0.957
	3.7	0.007	122400	25.87	0.5775	0.7	0.3	0.956
	3.7	0.005	129700	26.5	0.5875	0.7	0.3	0.962
	3.7	0.002	166500	29.05	0.66	0.7	0.3	1.008
	3.7	0.001	197000	31	0.905	0.7	0.3	1.183
	3.7	0.0008	217100	33.34	0.8075	0.7	0.3	1.110
	3.7	0.0005	307500	36.97	0.165	0.7	0.3	0.779
	3.7	0.0001	398800	41.23	0.4225	0.7	0.3	0.871
Kelso 3 (KLIP 3)	4.02	0.9999	10400	11.82	0.19	0.7	0.3	0.785
	4.02	0.99	11700	12.16	0.2075	0.7	0.3	0.789
	4.02	0.95	19000	13.7	0.2875	0.7	0.3	0.814
	4.02	0.9	25200	14.75	0.345	0.7	0.3	0.836
	4.02	0.8	33400	16.02	0.405	0.7	0.3	0.863
	4.02	0.7	38100	16.7	0.4325	0.7	0.3	0.876
	4.02	0.6	42800	17.39	0.455	0.7	0.3	0.887
	4.02	0.5	48000	18.13	0.475	0.7	0.3	0.898
	4.02	0.4	53200	18.81	0.4975	0.7	0.3	0.910
	4.02	0.3	60500	19.72	0.5225	0.7	0.3	0.924
	4.02	0.2	68800	20.7	0.54	0.7	0.3	0.934
	4.02	0.1	83400	22.27	0.5775	0.7	0.3	0.956
	4.02	0.05	99900	23.87	0.615	0.7	0.3	0.979
	4.02	0.04	104100	24.27	0.6225	0.7	0.3	0.984
	4.02	0.02	112600	25.22	0.62	0.7	0.3	0.982
	4.02	0.01	118200	25.82	0.6175	0.7	0.3	0.980
	4.02	0.007	122400	26.29	0.615	0.7	0.3	0.979
	4.02	0.005	129700	26.93	0.6275	0.7	0.3	0.987
	4.02	0.002	166500	29.52	0.7	0.7	0.3	1.034
	4.02	0.001	197000	31.5	0.8325	0.7	0.3	1.128
	4.02	0.0008	217100	33.39	0.7875	0.7	0.3	1.096
	4.02	0.0005	307500	37.33	0.215	0.7	0.3	0.791
	4.02	0.0001	398800	41.58	0.4575	0.7	0.3	0.888
Longview 2 (LVIP 2)	4.68	0.9999	10400	12.16	0.2475	0.7	0.3	0.801
	4.68	0.99	11700	12.51	0.265	0.7	0.3	0.806
	4.68	0.95	19000	14.08	0.3525	0.7	0.3	0.839
	4.68	0.9	25200	15.15	0.415	0.7	0.3	0.867
	4.68	0.8	33400	16.44	0.48	0.7	0.3	0.900

	Rating Curve from Calibrated Hydraulic Model				Uncertainty in Rating Curve			
					Natural	Sedimentation	Model	Total
	RM	Frequency	Discharge	Computed Water Surface	Sn	Ssed	Smodel	Stotal
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft)
	4.68	0.7	38100	17.13	0.515	0.7	0.3	0.919
	4.68	0.6	42800	17.81	0.54	0.7	0.3	0.934
	4.68	0.5	48000	18.55	0.5625	0.7	0.3	0.947
	4.68	0.4	53200	19.23	0.5875	0.7	0.3	0.962
	4.68	0.3	60500	20.13	0.6175	0.7	0.3	0.980
	4.68	0.2	68800	21.11	0.6425	0.7	0.3	0.996
	4.68	0.1	83400	22.67	0.685	0.7	0.3	1.024
	4.68	0.05	99900	24.25	0.735	0.7	0.3	1.058
	4.68	0.04	104100	24.66	0.74	0.7	0.3	1.062
	4.68	0.02	112600	25.59	0.7425	0.7	0.3	1.064
	4.68	0.01	118200	26.17	0.745	0.7	0.3	1.065
	4.68	0.007	122400	26.63	0.74	0.7	0.3	1.062
	4.68	0.005	129700	27.26	0.7525	0.7	0.3	1.071
	4.68	0.002	166500	29.86	0.85	0.7	0.3	1.141
	4.68	0.001	197000	31.85	0.745	0.7	0.3	1.065
	4.68	0.0008	217100	32.61	0.76	0.7	0.3	1.076
	4.68	0.0005	307500	37.3	0.5425	0.7	0.3	0.935
	4.68	0.0001	398800	41.56	0.5875	0.7	0.3	0.962
Longview 1 (LVIP 1)	4.9	0.9999	10400	12.36	0.2675	0.7	0.3	0.807
	4.9	0.99	11700	12.7	0.285	0.7	0.3	0.813
	4.9	0.95	19000	14.27	0.375	0.7	0.3	0.849
	4.9	0.9	25200	15.33	0.4425	0.7	0.3	0.881
	4.9	0.8	33400	16.6	0.5175	0.7	0.3	0.921
	4.9	0.7	38100	17.27	0.55	0.7	0.3	0.939
	4.9	0.6	42800	17.94	0.58	0.7	0.3	0.957
	4.9	0.5	48000	18.65	0.6075	0.7	0.3	0.974
	4.9	0.4	53200	19.31	0.6375	0.7	0.3	0.993
	4.9	0.3	60500	20.19	0.67	0.7	0.3	1.014
	4.9	0.2	68800	21.14	0.7	0.7	0.3	1.034
	4.9	0.1	83400	22.64	0.755	0.7	0.3	1.072
	4.9	0.05	99900	24.16	0.81	0.7	0.3	1.112
	4.9	0.04	104100	24.55	0.8225	0.7	0.3	1.121
	4.9	0.02	112600	25.44	0.825	0.7	0.3	1.123
	4.9	0.01	118200	26	0.8275	0.7	0.3	1.125
	4.9	0.007	122400	26.45	0.825	0.7	0.3	1.123
	4.9	0.005	129700	27.04	0.845	0.7	0.3	1.138
	4.9	0.002	166500	29.48	0.9675	0.7	0.3	1.231
	4.9	0.001	197000	31.33	1.0975	0.7	0.3	1.336

	Rating Curve from Calibrated Hydraulic Model				Uncertainty in Rating Curve			
					Natural	Sedimentation	Model	Total
	RM	Frequency	Discharge	Computed Water Surface	Sn	Ssed	Smodel	Stotal
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft)
	4.9	0.0008	217100	33.05	0.9825	0.7	0.3	1.243
	4.9	0.0005	307500	37.03	0.7175	0.7	0.3	1.046
	4.9	0.0001	398800	41.28	0.675	0.7	0.3	1.018
Kelso 2 (KLIP 2)	6.19	0.9999	10400	13.52	0.35	0.7	0.3	0.838
	6.19	0.99	11700	13.91	0.37	0.7	0.3	0.847
	6.19	0.95	19000	15.71	0.4475	0.7	0.3	0.883
	6.19	0.9	25200	16.93	0.505	0.7	0.3	0.914
	6.19	0.8	33400	18.36	0.57	0.7	0.3	0.951
	6.19	0.7	38100	19.12	0.6025	0.7	0.3	0.971
	6.19	0.6	42800	19.85	0.6325	0.7	0.3	0.990
	6.19	0.5	48000	20.63	0.66	0.7	0.3	1.008
	6.19	0.4	53200	21.37	0.685	0.7	0.3	1.024
	6.19	0.3	60500	22.34	0.7175	0.7	0.3	1.046
	6.19	0.2	68800	23.4	0.745	0.7	0.3	1.065
	6.19	0.1	83400	25.11	0.8	0.7	0.3	1.105
	6.19	0.05	99900	26.89	0.8425	0.7	0.3	1.136
	6.19	0.04	104100	27.34	0.8525	0.7	0.3	1.143
	6.19	0.02	112600	28.29	0.855	0.7	0.3	1.145
	6.19	0.01	118200	28.89	0.86	0.7	0.3	1.149
	6.19	0.007	122400	29.36	0.865	0.7	0.3	1.152
	6.19	0.005	129700	30.06	0.875	0.7	0.3	1.160
	6.19	0.002	166500	33.15	0.93	0.7	0.3	1.202
	6.19	0.001	197000	35.53	0.74	0.7	0.3	1.062
	6.19	0.0008	217100	36.07	0.715	0.7	0.3	1.045
	6.19	0.0005	307500	42.95	0.37	0.7	0.3	0.847
	6.19	0.0001	398800	48.12	-0.0925	0.7	0.3	0.767
Kelso 1 (KLIP 1)	7	0.9999	10400	14.24	0.4125	0.7	0.3	0.866
	7	0.99	11700	14.64	0.4375	0.7	0.3	0.878
	7	0.95	19000	16.53	0.535	0.7	0.3	0.931
	7	0.9	25200	17.81	0.6	0.7	0.3	0.970
	7	0.8	33400	19.29	0.6725	0.7	0.3	1.016
	7	0.7	38100	20.07	0.71	0.7	0.3	1.041
	7	0.6	42800	20.82	0.74	0.7	0.3	1.062
	7	0.5	48000	21.62	0.77	0.7	0.3	1.083
	7	0.4	53200	22.37	0.8	0.7	0.3	1.105
	7	0.3	60500	23.38	0.8325	0.7	0.3	1.128
	7	0.2	68800	24.46	0.8675	0.7	0.3	1.154

	Rating Curve from Calibrated Hydraulic Model				Uncertainty in Rating Curve			
					Natural	Sedimentation	Model	Total
	RM	Frequency	Discharge	Computed Water Surface	Sn	Ssed	Smodel	Stotal
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft)
	7	0.1	83400	26.23	0.9225	0.7	0.3	1.196
	7	0.05	99900	28.08	0.9725	0.7	0.3	1.235
	7	0.04	104100	28.54	0.985	0.7	0.3	1.245
	7	0.02	112600	29.51	0.9925	0.7	0.3	1.251
	7	0.01	118200	30.12	1.01	0.7	0.3	1.265
	7	0.007	122400	30.6	1.0175	0.7	0.3	1.271
	7	0.005	129700	31.32	1.03	0.7	0.3	1.281
	7	0.002	166500	34.72	1.0025	0.7	0.3	1.259
	7	0.001	197000	36.9	0.9875	0.7	0.3	1.247
	7	0.0008	217100	37.74	1.0325	0.7	0.3	1.283
	7	0.0005	307500	42.23	0.7725	0.7	0.3	1.085
	7	0.0001	398800	46.82	0.3875	0.7	0.3	0.854
Lexington 2 (LXIP 2)	8.3	0.9999	10400	15.97	0.4275	0.7	0.3	0.873
	8.3	0.99	11700	16.43	0.45	0.7	0.3	0.885
	8.3	0.95	19000	18.63	0.5325	0.7	0.3	0.929
	8.3	0.9	25200	20.14	0.5925	0.7	0.3	0.965
	8.3	0.8	33400	21.9	0.6625	0.7	0.3	1.009
	8.3	0.7	38100	22.82	0.695	0.7	0.3	1.031
	8.3	0.6	42800	23.7	0.725	0.7	0.3	1.051
	8.3	0.5	48000	24.64	0.7525	0.7	0.3	1.071
	8.3	0.4	53200	25.53	0.78	0.7	0.3	1.090
	8.3	0.3	60500	26.72	0.81	0.7	0.3	1.112
	8.3	0.2	68800	28	0.835	0.7	0.3	1.130
	8.3	0.1	83400	30.12	0.8775	0.7	0.3	1.162
	8.3	0.05	99900	32.34	0.9075	0.7	0.3	1.185
	8.3	0.04	104100	32.89	0.91	0.7	0.3	1.187
	8.3	0.02	112600	34.01	0.9175	0.7	0.3	1.192
	8.3	0.01	118200	34.72	0.925	0.7	0.3	1.198
	8.3	0.007	122400	35.25	0.9275	0.7	0.3	1.200
	8.3	0.005	129700	36.13	0.9275	0.7	0.3	1.200
	8.3	0.002	166500	40.28	0.8675	0.7	0.3	1.154
	8.3	0.001	197000	43.39	0.88	0.7	0.3	1.164
	8.3	0.0008	217100	45.98	0.64	0.7	0.3	0.995
	8.3	0.0005	307500	56.5	0.2125	0.7	0.3	0.791
	8.3	0.0001	398800	62.06	0.1875	0.7	0.3	0.784
ngt on 1 (LXI	8.64	0.9999	10400	16.52	0.4075	0.7	0.3	0.864
	8.64	0.99	11700	17.01	0.425	0.7	0.3	0.872

	Rating Curve from Calibrated Hydraulic Model				Uncertainty in Rating Curve			
					Natural	Sedimentation	Model	Total
	RM	Frequency	Discharge	Computed Water Surface	Sn	Ssed	Smodel	Stotal
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft)
	8.64	0.95	19000	19.34	0.4975	0.7	0.3	0.910
	8.64	0.9	25200	20.98	0.55	0.7	0.3	0.939
	8.64	0.8	33400	22.86	0.61	0.7	0.3	0.976
	8.64	0.7	38100	23.85	0.64	0.7	0.3	0.995
	8.64	0.6	42800	24.79	0.665	0.7	0.3	1.011
	8.64	0.5	48000	25.79	0.69	0.7	0.3	1.028
	8.64	0.4	53200	26.75	0.7125	0.7	0.3	1.043
	8.64	0.3	60500	28.03	0.7425	0.7	0.3	1.064
	8.64	0.2	68800	29.41	0.765	0.7	0.3	1.079
	8.64	0.1	83400	31.7	0.8025	0.7	0.3	1.106
	8.64	0.05	99900	34.12	0.83	0.7	0.3	1.126
	8.64	0.04	104100	34.72	0.835	0.7	0.3	1.130
	8.64	0.02	112600	35.93	0.8425	0.7	0.3	1.136
	8.64	0.01	118200	36.7	0.8475	0.7	0.3	1.139
	8.64	0.007	122400	37.28	0.85	0.7	0.3	1.141
	8.64	0.005	129700	38.24	0.85	0.7	0.3	1.141
	8.64	0.002	166500	42.8	0.795	0.7	0.3	1.101
	8.64	0.001	197000	46.16	0.735	0.7	0.3	1.058
	8.64	0.0008	217100	48.49	0.535	0.7	0.3	0.931
	8.64	0.0005	307500	57.23	0.2425	0.7	0.3	0.799
	8.64	0.0001	398800	62.63	0.2175	0.7	0.3	0.792
					0	0.7	0.3	
Castle Rock 3 (CRIP 3)	15.91	0.9999	10200	31.53	0.3075	0.25	0.3	0.497
	15.91	0.99	11500	32.14	0.34	0.25	0.3	0.518
	15.91	0.95	18600	35.05	0.4	0.25	0.3	0.559
	15.91	0.9	24700	36.99	0.435	0.25	0.3	0.585
	15.91	0.8	32900	39.2	0.49	0.25	0.3	0.627
	15.91	0.7	37500	40.32	0.5225	0.25	0.3	0.652
	15.91	0.6	42200	41.41	0.55	0.25	0.3	0.675
	15.91	0.5	47300	42.52	0.5825	0.25	0.3	0.701
	15.91	0.4	52400	43.59	0.61	0.25	0.3	0.724
	15.91	0.3	59600	45.02	0.645	0.25	0.3	0.754
	15.91	0.2	67800	46.53	0.6775	0.25	0.3	0.782
	15.91	0.1	82200	48.99	0.7225	0.25	0.3	0.821
	15.91	0.05	98600	51.51	0.765	0.25	0.3	0.859
	15.91	0.04	102700	52.1	0.765	0.25	0.3	0.859
	15.91	0.02	111000	53.26	0.76	0.25	0.3	0.854
	15.91	0.01	116400	53.94	0.7675	0.25	0.3	0.861
	15.91	0.007	120600	54.45	0.7625	0.25	0.3	0.857

	Rating Curve from Calibrated Hydraulic Model				Uncertainty in Rating Curve			
					Natural	Sedimentation	Model	Total
	RM	Frequency	Discharge	Computed Water Surface	Sn	Ssed	Smodel	Stotal
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft)
	15.91	0.005	127700	55.29	0.7775	0.25	0.3	0.870
	15.91	0.002	164200	59.46	0.7775	0.25	0.3	0.870
	15.91	0.001	194600	62.34	0.79	0.25	0.3	0.881
	15.91	0.0008	214700	64.07	0.7875	0.25	0.3	0.879
	15.91	0.0005	304900	70.89	0.7625	0.25	0.3	0.857
	15.91	0.0001	395800	76.16	0.7125	0.25	0.3	0.812
					0	0.25	0.3	
Castle Rock 2 (CRIP 2)	17	0.9999	10000	34.84	0.275	0.25	0.3	0.478
	17	0.99	11000	35.31	0.2975	0.25	0.3	0.491
	17	0.95	18000	37.99	0.3825	0.25	0.3	0.547
	17	0.9	24000	39.82	0.435	0.25	0.3	0.585
	17	0.8	32000	41.92	0.5	0.25	0.3	0.634
	17	0.7	36500	43	0.5375	0.25	0.3	0.664
	17	0.6	41000	44.03	0.5725	0.25	0.3	0.693
	17	0.5	46000	45.09	0.6075	0.25	0.3	0.722
	17	0.4	51000	46.1	0.64	0.25	0.3	0.750
	17	0.3	58000	47.45	0.68	0.25	0.3	0.784
	17	0.2	66000	48.88	0.72	0.25	0.3	0.819
	17	0.1	80000	51.2	0.7875	0.25	0.3	0.879
	17	0.05	96000	53.59	0.8525	0.25	0.3	0.938
	17	0.04	100000	54.16	0.8575	0.25	0.3	0.942
	17	0.02	108000	55.25	0.87	0.25	0.3	0.954
	17	0.01	113000	55.9	0.8875	0.25	0.3	0.970
	17	0.007	117000	56.4	0.89	0.25	0.3	0.972
	17	0.005	124000	57.22	0.9275	0.25	0.3	1.006
	17	0.002	160000	60.7	0.89	0.25	0.3	0.972
	17	0.001	190000	63.67	1.0425	0.25	0.3	1.113
	17	0.0008	210000	65.37	1.005	0.25	0.3	1.078
	17	0.0005	300000	72.25	0.9025	0.25	0.3	0.983
	17	0.0001	390000	77.61	0.835	0.25	0.3	0.922
Castle Rock 1 (CRIP 1)	17.42	0.9999	10000	35.7	0.3025	0.25	0.3	0.494
	17.42	0.99	11000	36.2	0.325	0.25	0.3	0.508
	17.42	0.95	18000	39.06	0.4125	0.25	0.3	0.568
	17.42	0.9	24000	41.02	0.4625	0.25	0.3	0.605
	17.42	0.8	32000	43.25	0.53	0.25	0.3	0.658
	17.42	0.7	36500	44.38	0.565	0.25	0.3	0.687
	17.42	0.6	41000	45.46	0.6	0.25	0.3	0.716
	17.42	0.5	46000	46.59	0.635	0.25	0.3	0.745

Rating Curve from Calibrated Hydraulic Model				Uncertainty in Rating Curve			
				Natural	Sedimentation	Model	Total
RM	Frequency	Discharge	Computed Water Surface	Sn	Ssed	Smodel	Stotal
		(cfs)	(ft)	(ft)	(ft)	(ft)	(ft)
17.42	0.4	51000	47.66	0.665	0.25	0.3	0.771
17.42	0.3	58000	49.08	0.705	0.25	0.3	0.806
17.42	0.2	66000	50.59	0.745	0.25	0.3	0.841
17.42	0.1	80000	53.03	0.805	0.25	0.3	0.895
17.42	0.05	96000	55.53	0.865	0.25	0.3	0.949
17.42	0.04	100000	56.12	0.875	0.25	0.3	0.958
17.42	0.02	108000	57.27	0.8875	0.25	0.3	0.970
17.42	0.01	113000	57.95	0.905	0.25	0.3	0.986
17.42	0.007	117000	58.48	0.91	0.25	0.3	0.990
17.42	0.005	124000	59.35	0.9425	0.25	0.3	1.020
17.42	0.002	160000	63.08	0.81	0.25	0.3	0.899
17.42	0.001	190000	65.54	0.885	0.25	0.3	0.967
17.42	0.0008	210000	66.99	0.695	0.25	0.3	0.797
17.42	0.0005	300000	72.21	1.03	0.25	0.3	1.102
17.42	0.0001	390000	77.48	1.1275	0.25	0.3	1.193

Appendix D. Cowlitz River Levees Safe Water Level Study

D.1. ADDENDUM

This addendum updates the Safe Water Level (SWL) for Castle Rock reach CR8 and updates the levee system response curves for the Lexington and Coweeman levees. These items required updating as the values provided in the March 2009 SWL study were considered too conservative after a Portland District Engineering and Construction Division review.

Castle Rock
The Castle Rock levee in reach CR8 at the downstream end of the levee, by the City's sewage treatment plant, was widened and raised after Corps construction in 1956. As shown in Photo 2.8a in report, the levee section is now broad in this reach. The SWL for this reach has been set at the top of the levee so that the reach is now an overtopping only failure reach.

Lexington and Coweeman
As described on page 4 of the March 2009 SWL study, a simplified approach was used to develop system response curves for the levees above the SWL. A straight-line relationship was used between two points: probability of failure = 0 at the SWL and probability of failure = 1 at the levee top. This simplified approach lead to overly conservative estimates of the levee response above the SWL. The critical potential failure mode for Lexington and Coweeman levees is underseepage. Additional analyses have been performed to refine the system response curves for these two levees.

A pseudo-reliability approach was used to develop the updated system response curves. The approach is based on the character of the distribution of seepage exit gradient for underseepage problems. As described in Griffiths and Fenton (1998), the distribution of the exit gradient is lognormal. Figure A-1 is a figure from the reference showing an example distribution. Griffiths and Fenton performed reliability analyses with a range of values of variability in the permeability of an example foundation. Because the distribution of exit gradient is lognormal with a skew to the right (higher exit gradients), the authors were able to conclude the following: "It should be noted that irrespective of the θ_k {measure of spatial correlation of permeability, k} or COV_k {coefficient of variation of permeability, k}, $P[i_e > i_{det}]$ {probability that the exit gradient is greater than the deterministic exit gradient} is always less than 50%. This is a reassuring result from a design standpoint." In other words, since the distribution of exit gradient is lognormal, the mean exit gradient from a deterministic approach using mean input values is always greater than the median exit gradient, at which 50% of values in distribution are above and 50% of values are below. The bottom line is that if the exit gradient is computed deterministically with most likely values, the probability that the exit gradient will be greater than the deterministic value is always less than 50%.

This fact was employed as follows. Seepage analyses were performed to determine the river stage at which the exit gradient equals the critical exit gradient. At this river stage, due to the fact above, the probability that the exit gradient is above the critical gradient – i.e. the probability of failure – is less than 50%. 50% is the upper bound. For the purposes of this analysis, a conservative value of 50% was used for the probability of failure when the deterministic exit gradient equals the critical gradient.

Another way to think about this is as follows. If the distribution of exit gradient were symmetrical about the mean, then if we calculated a mean exit gradient exactly equal to the critical gradient, the probability of failure and the probability of nonfailure would each be 50%. Since the exit gradient distribution is

lognormal, skewed to the right, the probability of failure is less than 50% and the probability of nonfailure is greater than 50%. Using 50% for the probability of failure is conservative.

Figures A-2 and A-3 show how this was done for Lexington and Coweeman. The river stage at which the deterministic exit gradient equals the critical gradient was determined and the probability of failure due to underseepage was assumed equal to 50% at this stage. For example, for station 47+20 on Lexington levee, the deterministic exit gradient equals the critical gradient of 0.92 at a river stage of 48.4 ft. The underseepage response curve goes from probability of failure = 0 at the SWL to probability of failure = 0.5 at elevation 48.4 ft, which is approximately the top of levee. The overall system response curve follows the seepage curve up to the top of levee, then the probability of failure jumps to 1.0 due to overtopping.

Updated Index Point System Response Curves

The index point values below take the place of those shown in the March 2009 report.

Index Point	River Elevation (NAVD88)	Probability of Failure
CRIP3	58.5	1
	The probability of failure = 0 up to top of levee	
LXIP1	38.2	0
	45.7	0.5
	45.7+	1
CWIP1*	25.0	0
	28.3	0.25
	28.3+	1
CWIP2*	25.0	0
	27.4	0.25
	27.4+	1

*The river stage at which probability of failure = 0.5 for underseepage is above top of levee (see extrapolated line in Figure A-3). Using extrapolated line, probability of failure = 0.25 at top of levee.

Reference

Griffiths, D. V. and Fenton, G. A. (1998). *Probabilistic Analysis of Exit Gradients Due to Steady Seepage*, ASCE Journal of Geotechnical and Geoenvironmental Engineering, pages 789 – 797, Vol. 124, No. 9.

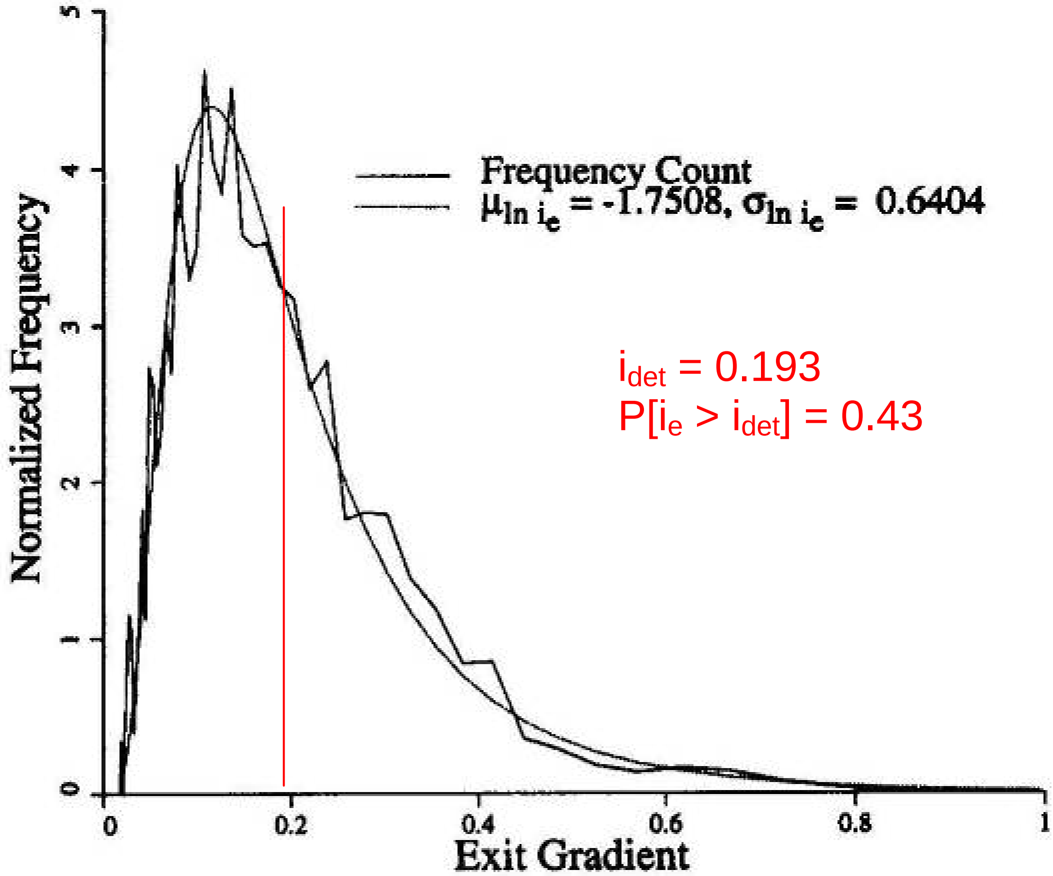
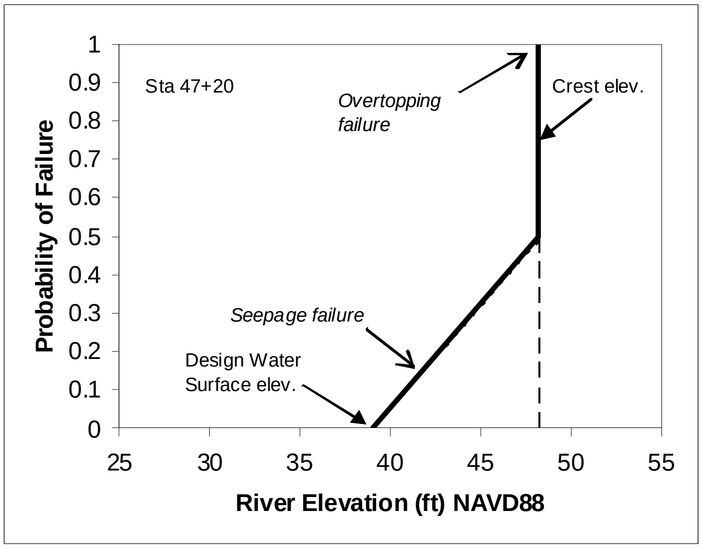
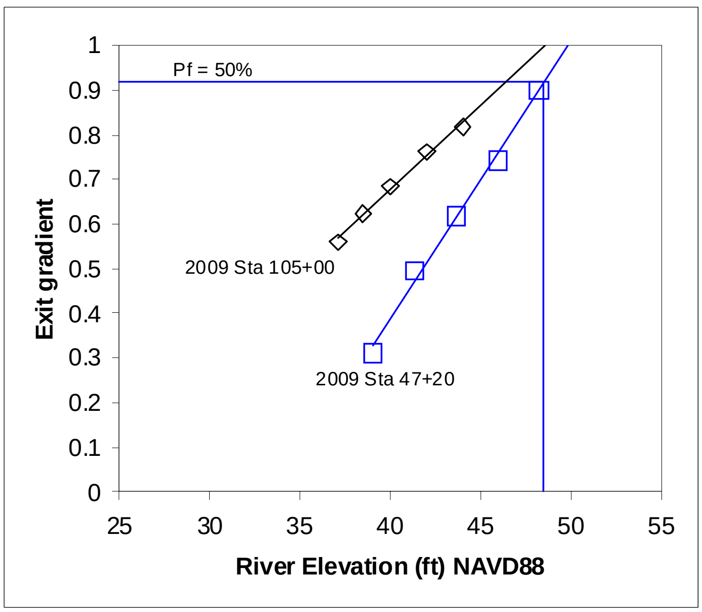


FIG. 17. Histogram of Exit Gradients in 2D for Case $\theta_k = 2$ m and $COV_k = 1$

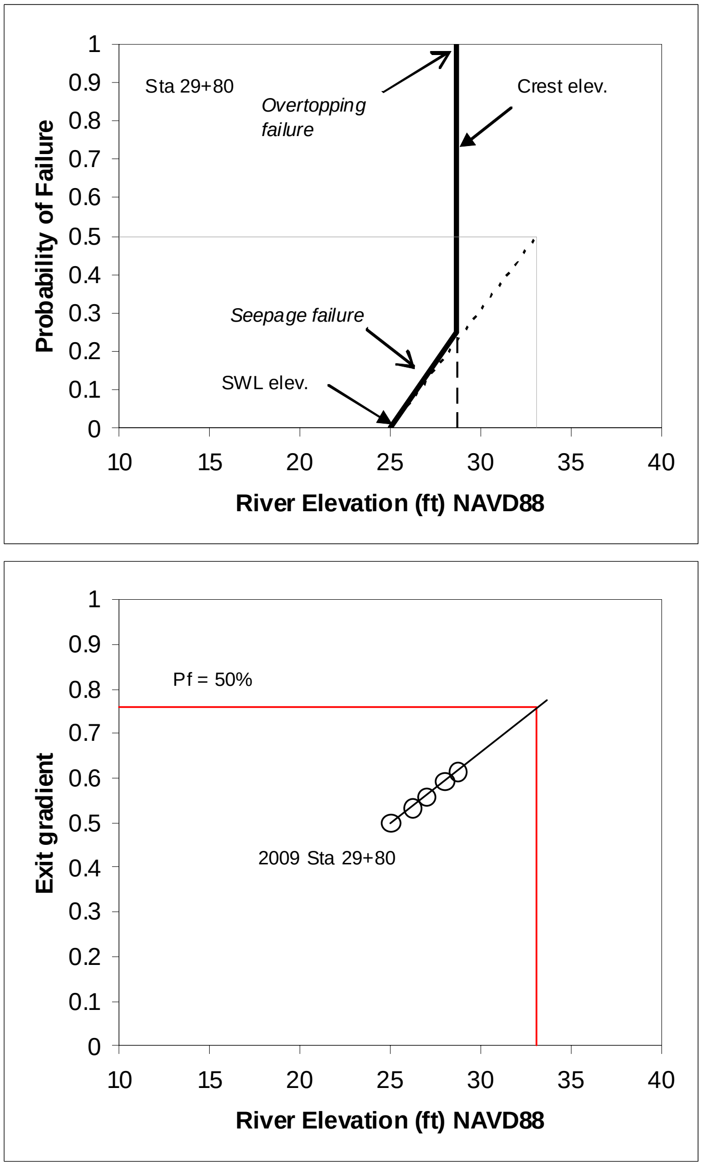
Figure D. 1. Example distribution of exit gradient for underseepage analysis, from Griffiths and Fenton (1998)





For toe saturated unit weight = 120 pcf
Critical exit gradient = 0.92
FS = 1 at critical exit gradient
Assume Pf = 50% when FS = 1

Figure D. 2. Lexington Levee



For blanket saturated unit weight=110 pcf
Critical gradient across blanket = 0.76
FS = 1 at critical gradient
Assume Pf = 50% when FS = 1

Figure D. 3. Coweeman Levee

D.2. INTRODUCTION

D.3. PURPOSE

The Corps of Engineers is authorized by Congress to maintain specified levels of flood protection along the Cowlitz River for Castle Rock, Lexington, Longview, and Kelso (North and South). Figure 1.1 shows the locations of these levees. The Level of Protection (LoP) for these areas was updated in 2008; it was determined that the LoP's for Castle Rock, Lexington, and South Kelso are below the authorized levels, and the authorized LoP's for North Kelso and Longview are met.

The two main factors causing the reduction in LoP's are increased sediment deposition in the Cowlitz River and an updated estimate of the Cowlitz River's flow-frequency relationship. Since the Sediment Retention Structure (SRS) on the North Fork Toutle River below Mount St. Helens is now a run-of-the-river project (there is no longer a settling pool behind the structure; all flow passes the spillway), more sand is passing the structure and depositing in the Cowlitz River, raising the river bed and decreasing channel conveyance. A re-analysis of the watershed's hydrology, including the hydrologic data obtained since the last hydrology analysis in 1997, resulted in an increased estimate of flow for a given frequency event. For example, the 100-year flow at Castle Rock is 116,000 cfs now compared to the 97,000 cfs estimated in 1997, an increase of 20 percent. The 2008 LoP's, influenced by the two factors described above, are presented in the *Mount St. Helens Project Cowlitz River Levee Systems Level of Flood Protection Update* (Portland District 2008 draft).

For the 2008 LoP update, the reliability of the levees, represented by geotechnical system response curves, was based primarily on the levee geotechnical reliability evaluation presented in the *Mount St. Helens Engineering Reanalysis; Hydrologic, Hydraulics, Sedimentation and Risk Analysis; Design Documentation Report* (Portland District 2002). The estimated reliability of the Lexington levee was increased, compared to the 2002 reliability, based on post-2002 analyses.

The purpose of the study described in this report is to update the evaluations of the levees' safe water levels and geotechnical system response curves for use in future LoP determinations. The study includes the Coweeman levee. The Portland District is currently updating the long-term plan to manage sediment from Mount St. Helens and maintain authorized LoP's. The geotechnical system response curves presented in this report will be used in that effort.

D.4. EVALUATION METHOD

After reviewing the construction history, inspection history, available reports and drawings, survey data, and exploration program results, and making a site visit, each levee was divided into reaches of similar characteristics. The reaches were then screened to determine which ones required seepage and stability analyses, as described below. Based on judgments made during screening, and the results of the seepage and stability analyses, Safe Water Levels were determined. The sections below for each levee include plots of Safe Water Level and top of levee. Survey data from 2007 was used for the top of levee elevations.

Safe Water Levels (SWLs) represent the highest flood level for which reasonable assurance can be made that the levee will not fail. This level is further defined as the river stage at which only normal surveillance and minor remedial work will be required during normal flood periods and close surveillance during extended periods.

There are design water surfaces for the Corps levees on the Cowlitz and Coweeman Rivers. The approach used in this study was to start with the assumption that the SWL is the Design Water Surface (DWS). Analyses of various failure modes were performed to verify this assumption for each levee. Table 1.1 shows the requirements for a "safe" condition for the failure modes considered. As determined by the analyses, the SWL was either raised or lowered in relation to the DWS. The performance of the levees during past flood events was also considered. If a levee has been tested by a flood event higher than the DWS (such as the flood of 1996 for the Coweeman levee), the levee performed safely during the event, and the levee conditions have not changed (e.g. due to trees or other significant encroachments in the levee), it is expected that the levee should be safe for a future flood event of similar stage and duration. The ability of the diking district to respond as in the previous event is also a consideration.

Table 1.1. Requirements for SWL

Failure mode	Requirement for “safe” condition	Reference
Slope instability due to steady seepage	$FS^{(1)} \geq 1.4$	EM 1110-2-1913, Chapter 6
Erosion & piping due to underseepage	$i_e^{(2)} \leq 0.5$	EM 1110-2-1913, Appendix B
Erosion due to river current	Adequate slope protection such as riprap[3]	EM 1110-2-1913, Chapter 7
Overtopping	No overtopping	

Notes:
(1) FS = Factor of Safety
(2) i_e = exit gradient
(3) For this study the size of the riprap was not re-evaluated for expected river velocities. The presence of riprap as shown on as-constructed drawings was checked.

Seepage and stability analyses were performed using the computer programs SEEP/W (finite element method) and SLOPE/W (limit equilibrium method) in GeoStudio 2007. All seepage analyses were for steady-state seepage conditions. With the absence of substantial zones of low permeability fine-grained soils, the presence of which would reduce the time required to reach steady-state conditions, it was considered too unconservative to not assume steady-state conditions.

The Coweeman levee does have a low permeability landside foundation upper “blanket” layer, a silt (MH). However, the foundation layer below is a higher permeability sand with silt and some gravel. During the 1996 flood, boils occurred at the levee toe within one to four days after the river rose. During the recent January 2009 event, which was shorter in duration than the 1996 event, levee through-seepage occurred near the upstream end of the levee by the RV park. In the spring, high water in the Columbia River can potentially back into the Coweeman for a month or longer. Given these considerations, a steady-state seepage analysis for the Coweeman levee was considered appropriate.

Steady-state seepage pore water pressures from the SEEP/W analyses were used in the SLOPE/W slope stability analyses. The factor of safety against slope failure was computed using Spencer’s method. Slip surfaces were required to intersect the crest of the levee and be at least 5 ft deep at one point along the slip surface. It is assumed that each active diking district would be able to buttress the levee if a shallower slide occurs, to prevent the levee from unraveling. In addition, slip surfaces in which steeply placed, temporary quarry waste material slides over the permanent levee section were not considered critical.

Soil properties for seepage and stability analyses were estimated primarily from the *Geotechnical Data Report, Cowlitz River Levees Investigation* (Cornforth Consultants, 2001). Cornforth performed drilling and SPT sampling, laboratory tests (water content, Atterberg limits, particle-size analysis, and two direct shear tests), in-situ falling head tests, and cone penetrometer tests. Information from earlier investigations (e.g. 1956 Castle Rock subsurface investigation and 1961 Coweeman subsurface investigation) and construction history was also used. As-constructed drawings, cross-section surveys from 2001, and boring log information were used to develop levee and foundation sections for the analyses.

As shown in Figure 1.2, a levee is considered safe for river levels up to the SWL, as requirements for a safe condition are met. For river levels above the SWL, the requirements for a safe condition are not met, so the stability of the levee is uncertain. *This does not mean that the levee is predicted to fail for river levels above the SWL. It means the levee stability is unreliable.* For river levels overtopping the levee, failure is considered likely.

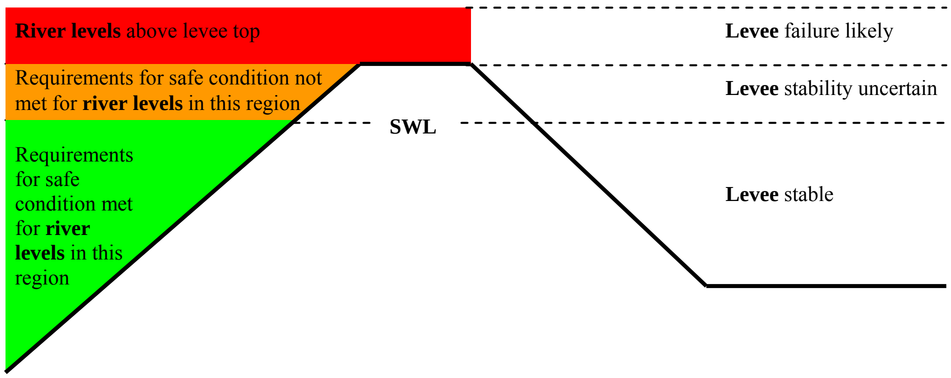


Figure 1.2. Representation of Safe Water Level

The SWLs established in this report can be used to develop simplified geotechnical system response curves for use in Level-of-Protection (LoP) evaluations. The simplified curve is a straight line between probability of failure (Pf) = 0 at the SWL and probability of failure = 1 at the levee crest. In this analysis the Pf = 1 profile is taken as the current top elevation, which includes temporary raises placed in the early 80s (e.g. quarry waste) that have not been removed. While these temporary raises do not increase the SWLs, they do provide some degree of overtopping protection. By setting Pf = 1 at the top of a temporary raise, some benefit is given to the material to protect against overtopping; for river stages against the temporary raises, the Pf is not quite 1 but is very high.

In the sections below for each levee, index points are provided for LoP evaluations. Each index point includes a levee station that can be correlated to a river mile, and a simple geotechnical system response curve composed of either one or two river stages.

The index points were chosen assuming that the diking districts will raise certain known low spots as a flood event is approaching. If this assumption becomes invalid for a low spot, it may be appropriate to locate an index point at the location. The low spots are listed in the table below.

Table 1.2. Low spots to raise temporarily by diking districts as flood event approaches

	Station
Castle Rock Levee	
Just downstream of bridge	28+00
Road crossing by sewage treatment plant	60+00
Lexington Levee	
Westside Hwy crossing	20+45
McCorkle Creek pump station	42+00
A-frame house	110+50
Longview Levee	
Railroad crossing	11+30
Driveway (1 of 2 in the vicinity)	15+90
Fishers Lane crossing	22+50
Hall of Justice area	41+20 to 57+00
Near intersection of Hwy 411 and Peardale Lane	71+00
Apartments near River Rd. and Hudson St.	81+50
Near north end of Marine View Drive	119+00
Kelso	
N. Kelso	
Railroad tunnel	Upstream end
Cowlitz Gardens Road	25+50
Concrete flood wall	29+00
S. Kelso	
Rotary Landing	113+70
Mill Street	142+60
Coweeman Levee	
South Kelso Drive	72+20
Grade Street	86+60

D.5. CASTLE ROCK LEVEE

D.5.1. Information Reviewed

D.5.1.1. Construction History

Located on the left bank of the Cowlitz River at Castle Rock, Washington, the levee protection surrounding the city was destroyed during the flood of December 1933. Reconstruction of the levee began in 1934 with increased levee sections and a crest top elevation with limited freeboard above the 1933 flood stage. The Flood Control Act approved August 18, 1941 included provisions for flood-control improvements to the Castle Rock levee to protect the city from floods of a magnitude equal to the maximum flood of record that occurred in December 1933. Planning and contract documents preparation by the Corps took place between 1955 and 1956 for the then authorized project which would raise the levee to 3 feet above the 1933 flood level, place revetment at the upstream and downstream reaches of the levee, add several retaining walls for existing structures, and install or extend three gravity drain pipes. Construction of the authorized project was completed in December 1956.

After the 1980 Mount St. Helens Eruption and flood, additional protection for the upstream portion of the levee was evaluated, and a construction contract was awarded in 1980. Additional levee embankment and revetment was placed for the then anticipated design flood water surface plus 3 feet. An additional 2 foot overbuild with a 12-foot wide access road top was included in this build. As sediment was transported downstream from the watershed surrounding Mt Saint Helens into the Cowlitz River, expected new flood stage elevations were calculated that resulted in an emergency levee raise situation in 1982 to provide temporary increased protection for the Castle Rock levee and downstream Cowlitz River levees. Temporary levee raise measures along the length of the levee were completed in early1983. Some of the temporary raise elements were later removed. Some improvements were constructed after 1999 by the City of Castle Rock on the levee top to provide a paved trail for recreational access.

D.5.1.2. Inspection History

Inspections of the levee took place in 1969, 1973, and then continued on an annual basis through 1977. No inspections were recorded between 1978 and 1983. After the 1980 eruption and levee raises in 1980, other levee work in 1982 and 1983, annual inspections resumed and have been conducted since then. Continued maintenance has kept the levee eligible for the rehabilitation and inspection program and the levee continues to remain active in the program.

D.5.1.3. Reports and Drawings

The following reports and drawings are available:

- February 1956 Design Memorandum for the Castle Rock "Levee Raising and Strengthening" that includes limited soil boring and testing information, plan, profile, and sections of proposed work
- As-Constructed Drawings for the 1956 Castle Rock Levee Improvement Contract
- Contract Drawings for the 1980 Mount St. Helens Eruption levee raise work
- As-Constructed Drawings for the 1982 Cowlitz River Levee Improvement for the Castle Rock Levee
- Report that included Castle Rock Levee in the "Cowlitz-Coweman Levee Final Report" completed by AE contractor Ogden Beeman & Associates in November 1985 for proposed levee improvements to meet 100-, 500-, and 500-year plus 4 feet protection
- Mount St. Helens Engineering Reanalysis, Hydrologic, Hydraulics, Sedimentation, and Risk Analysis Design Documentation Report, April 2002 that included analysis for the Castle Rock Levee

D.5.1.4. Surveys

Surveys for the levee top profiles include the 1980 pre-construction survey, 1982 As-Built survey, 2001 survey, 2007 National Levee Inventory survey (vertical accuracy of approximately 2 cm), and the 2007 LIDAR survey. There are cross sections from the 2001 and 2007 surveys at selected locations.

D.5.1.5. Exploration Programs

There is limited subsurface information for the Castle Rock levee. For the 1956 Design Memorandum, 19 shallow auger holes were drilled in the existing levee and a limited number of soil samples were sent to the laboratory for grain size testing, permeability tests, and soil density estimates. The only other explorations involved 4 borings with in-place SPT tests and 1 CPT (Cone Penetrometer Test) probe during the 2001 program at Castle Rock, and a few field permeability tests. Soil properties for the embankment and foundation soils were estimated from the 2001 explorations.

D.5.2. Castle Rock Levee Reaches

Figure 2.1 shows the division of the Castle Rock Levee into 8 reaches, CR1 – CR8. Figure 2.2 is a profile of the levee top and safe water level. Table 2.1 shows the station endpoints of each reach.

Table 2.1. Castle Rock Levee reaches

Reach	Station (2007 levee inventory station)		
CR1	0+00	to	4+80
CR2	4+80	to	16+60
CR3	16+60	to	22+80
CR4	22+80	to	25+50
CR5	25+50	to	26+70
CR6	26+70	to	34+20
CR7	34+20	to	59+00
CR8	59+00	to	End

D.5.2.1.Reach CR1

Reach CR1 extends from the upstream end of the levee to approximately station 4+80. There is approximately 3 – 4 ft of quarry waste material that was placed as a temporary raise in 1982 and has not been removed. The interior ground elevation is relatively high in this reach, as shown in Figure 2.2.

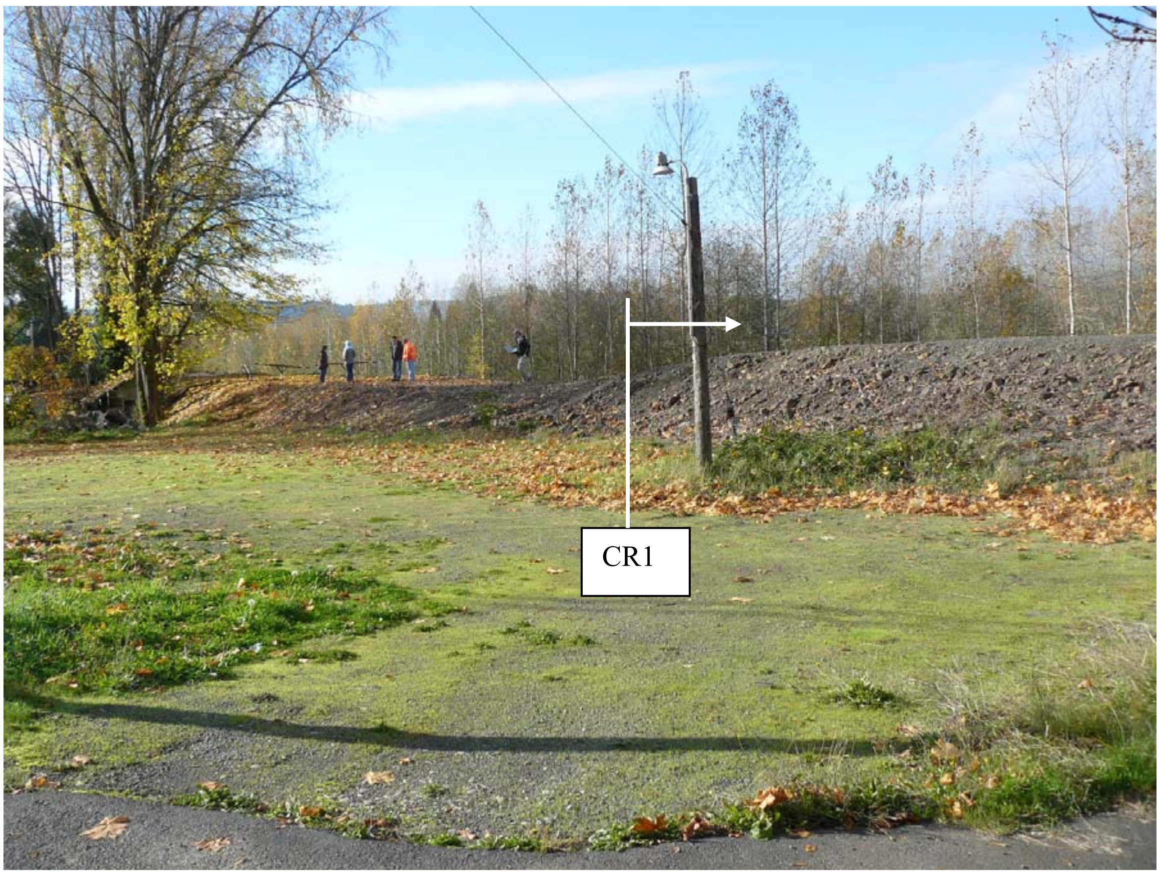


Photo 2.1. Castle Rock reach CR1

D.5.2.2.Reach CR2

Reach CR2 extends from approximately station 4+80 to station 16+60. Instead of quarry waste, plywood crib structures were used to temporarily raise this stretch of levee in 1982. Reach CR2 has house structures and large trees very close to, and in, the backslope of the levee.

Photo 2.2. Castle Rock reach CR2

D.5.2.3. Reach CR3

Reach CR3 extends from approximately station 16+60 to station 22+80. There is approximately 3 – 4 ft of quarry waste material that was placed as a temporary raise in 1982 and has not been removed.

Photo 2.3. Castle Rock reach CR3

D.5.2.4.Reach CR4

Reach CR4 extends from approximately station 22+80 to station 25+50. This is another stretch where plywood crib structures were used to temporarily raise the levee in 1982.

Photo 2.4. Castle Rock reach CR4

D.5.2.5.Reach CR5

Reach CR5 is the Arkansas Valley Road Bridge.

Photo 2.5. Castle Rock reach CR5

D.5.2.6.Reach CR6

Reach CR6 extends from approximately station 26+70 to station 34+20. This reach was constructed prior to 1980; it does not sit on the 1980 mudflow (sand) and it has a less steep backslope than the levee north of the bridge.

Photo 2.6. Castle Rock reach CR6

D.5.2.7.Reach CR7

Reach CR7 extends from approximately station 34+20 to station 59+00. There is a large dredge spoil area riverward of the levee protecting this reach. The elevation of the spoil area is similar to the top elevation of the levee.

Photo 2.7. Castle Rock reach CR7

D.5.2.8.Reach CR8

Reach CR8 extends from approximately station 59+00 to the downstream end of the levee. This reach used to include a spillway section to control overtopping, but the section has been raised to protect the sewage treatment plant. The levee section is broad, as can be seen in the photo below.

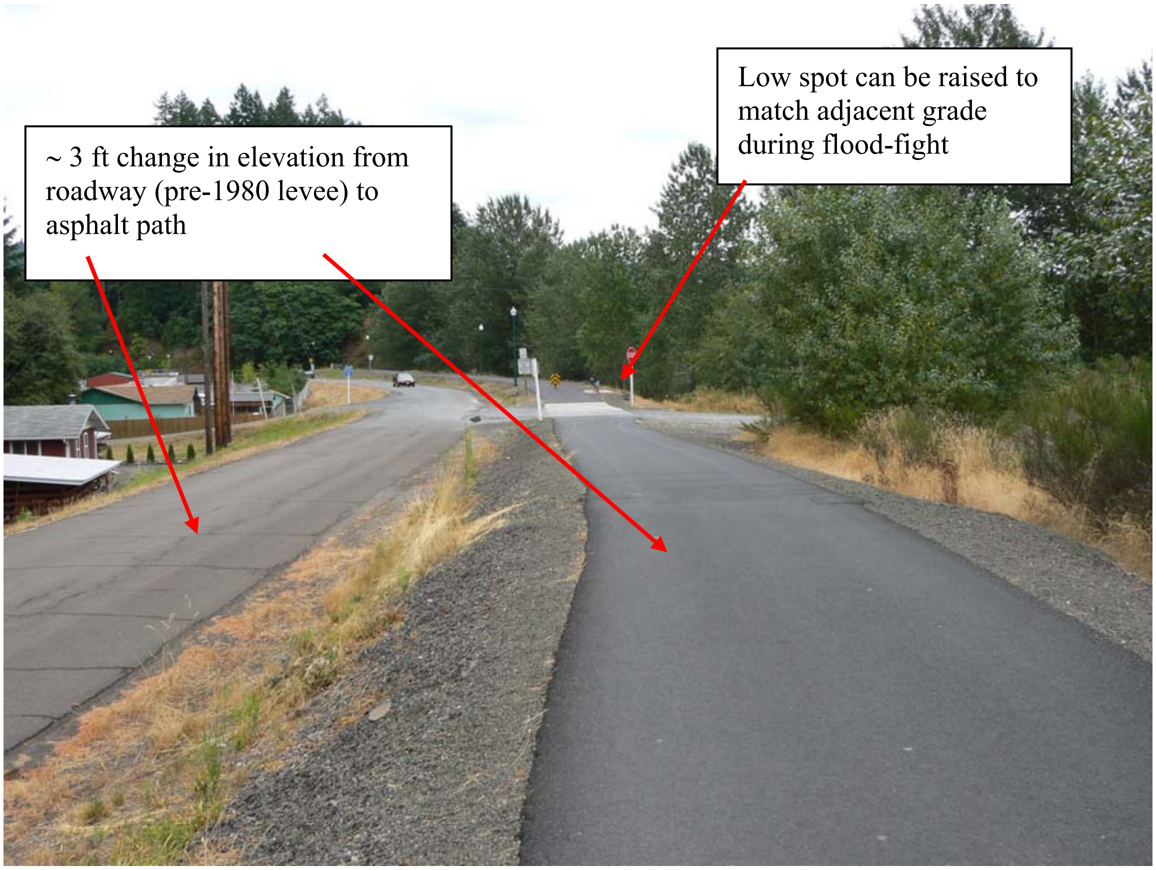


Photo 2.8 a. Castle Rock reach CR8

Photo 2.8 b. Castle Rock reach CR8, riverward slope with riprap to levee crest

D.5.3. Levee Reaches Screening

In 1980, after the eruption of Mount St. Helens, a new levee was constructed north of the Arkansas Valley Road Bridge (reaches CR1 – CR4). Due to real estate constraints, the new levee was constructed riverward of the existing, smaller levee. This resulted in the riverward half of the new levee being founded on the elevated river bottom resulting from the eruption: the loose, poorly graded fine sand with about 5% low-plasticity fines. Drawing CZG-5-1/4 shows the elevated river bottom. Boring log CR-DH-01 shows the sediment, which is 8.5-ft-thick in this location. This permeable deposit lets high water pressures develop beneath the levee when the river is up. Another result of the real estate constraints is that the backslope of the 1980 levee is relatively steep (2H:1V to 1.5H:1V). Both these factors—the permeable sand beneath the riverward half of the levee and the steep backslopes—result in a short seepage path beneath the levee along which most of the head drop occurs when the river is high. During very high river stages there is the potential for quick conditions to develop near the levee toe.

The conditions described above are not as much of a concern for reaches CR1 and CR4 because of the relatively high interior ground elevations in these reaches. The safe water levels for these reaches were based on the interior levee toe elevations. Further analyses were performed for reaches CR2 and CR3.

Conditions are more favorable south of the bridge. In reach CR6, the interior toe elevation is about 2 ft higher, on average, than the interior toe elevation where the 1980 DWS applies in reaches CR2 and CR3.

Given the less-steep backslope in CR6, the SWL for reach CR6 was taken as the downstream projection of the 1980 DWS plus 2 ft.

In reach CR7, the safe water level is the top of the levee as a result of the dredge spoil material riverward of the levee.

Figure 2.2 shows the 1956 DWS as interpreted from Drawing CZ-2-8/1 and the following paragraph from the 1956 Design Memorandum for the levee:

> "The basic design criteria adopted for flood protection of the Castle Rock area on Cowlitz River contemplates reasonable protection against a flood of 1933 magnitude. Freeboard of 3 feet would be provided over the water-surface profile for that flood."

The 1956 DWS shown in Figure 2.2 is the design grade (top of levee) shown in Drawing CZ-2-8/1 minus 3 ft of freeboard. Where the 1956 DWS appears to drop dramatically in reach CR8, seepage and stability analyses were performed for the levee section in this reach.

D.5.4. Castle Rock Levee Seepage and Stability Analyses

Seepage and stability analyses were performed for the levee at stations 10+00 (reach CR2), 20+00 (reach CR3), and 63+00 (reach CR8).

D.5.4.1. Station 10+00 (reach CR2)

Figures 2.3 and 2.4 show the results of the seepage and stability analyses for the levee at station 10+00. Table 2.2 shows the soil properties used in the analyses. With the river at the 1980 DWS, the seepage exit gradient at the toe is 0.62 and the landward slope factor of safety against sliding is 1.6.

Table 2.2. Reach CR2, station 10+00 soil properties

	Quarry waste	Levee sand with gravel	1980 Foundation sand	Foundation gravel and sand with silt
Permeability, k (ft/min)	0.1	0.002	0.02	0.002
Horiz. k / Vert. k	4	4	4	4
Unit weight (pcf)	135	140	120	130
c' (psf)	0	0	0	0
ϕ' (degrees)	40	40	34	37

D.5.4.2. Station 20+00 (reach CR3)

Figures 2.5 and 2.6 show the results of the seepage and stability analyses for the levee at station 20+00. Table 2.3 shows the soil properties used in the analyses. With the river at the 1980 DWS, the seepage exit gradient at the toe is 0.43 and the landward slope factor of safety against sliding is 1.6.

Table 2.3. Reach CR3, station 20+00 soil properties

	Quarry	Levee sand	Drain rock	1980	Foundation

	waste	with gravel		Foundation sand	sand with silt and gravel
Permeability, k (ft/min)	0.1	0.002	1	0.02	0.002
Horiz. k / Vert. k	4	4	1	4	4
Unit weight (pcf)	135	140	110	120	130
c' (psf)	0	0	0	0	0
ϕ' (degrees)	40	40	30	34	37

D.5.4.3.2.4.3 Station 63+00 (reach CR8)

Figures 2.7 and 2.8 show the results of the seepage and stability analyses for the levee at station 63+00. Table 2.4 shows the soil properties used in the analyses. With the river at elevation 55.5 ft (the SWL shown in Figure 2.2), the seepage exit gradient at the toe is 0.58 and the landward slope factor of safety against sliding is 1.9.

Table 2.4. Reach CR8, station 63+00 soil properties

	Quarry waste	Levee	Foundation
Permeability, k (ft/min)	0.1	0.002	0.002
Horiz. k / Vert. k	4	4	4
Unit weight (pcf)	135	137	125
c' (psf)	0	0	0
ϕ' (degrees)	40	37	36

D.5.5. Discussion

In reaches CR2 and CR3, the SWL is taken as the 1980 DWS where the interior toe elevation is relatively low (most of reach CR2 and all of reach CR3). The fact that the computed seepage exit gradient at station 10+00, 0.6, is just above the safe value, 0.5, leads to the conclusion that the SWL should not be raised above the 1980 DWS.

The levee section at station 63+00 is quite broad, as shown in Figures 2.7 and 2.8. Given the substantial width of the levee, it seems reasonable that the levee should be safe for river levels up to the elevation of the road on the landward side of the levee. The fact that the computed seepage exit gradient is close to (just above) the safe value helps justify this assumption. The SWL for reach CR8 is taken as the elevation of the landside road, which is approximately 3 ft below the crest of the levee. This SWL is above the 1956 DWS.

D.5.6. Castle Rock Index Points

The Castle Rock levee has three index points, designated Castle Rock Index Point (CRIP) 1 – CRIP 3. The low spots in reaches CR6 (station 28+00) and CR8 (station 60+00) were not used as index points because it is assumed that the City can temporarily raise these sections to match the adjacent grades during a flood event.

Table 2.5. Castle Rock Index Points

Index Point	Station (2007 survey stationing)	Elevation (NAVD88) at which:	
		Probability of failure = 0	Probability of failure = 1
CRIP1*	10+00	57.4	61.5
CRIP2	32+14	57.3	60.9
CRIP3**	63+00	55.5	58.5

POST-REPORT COMMENTS:
*** Pf = 0 up to top of levee for this index point after installation of seepage cutoff wall in 2009.**
**** See Addendum for updated fragility curve points.**

D.6. LEXINGTON LEVEE

D.6.1. Information Reviewed

D.6.1.1. Construction History

The Lexington dike prior to 1933 had been built to a crest elevation that varied from approximately elevation 29.3 at the upstream end to elevation 27.3 at the downstream end. The dike tied into adjacent high ground at both ends. A short length of high ground at the half way point of the dike did not require a dike section. In 1973 a proposed levee raise for the Lexington dike was presented to Cowlitz County. Construction of the levee raise was accomplished soon after.

After the 1980 eruption, a Corps designed levee raise was constructed in 1980 and 1981. A temporary levee raise was constructed in 1982 by the Corps and most elements are still in place. Lexington Flood Control Zone Levee is classified as being on the Non-Federal Levee Program.

D.6.1.2. Inspection History

The first annual inspection was made in 1982 and continued through 1986. In 1988 the annual levee maintenance inspection for the levee to remain as an active participant in the rehabilitation program first took place. Annual inspections have continued since 1988. The levee remains active in the rehabilitation and inspection program.

D.6.1.3. Reports and Drawings

The following reports and drawings are available:

- 1973 Report and drawings for a proposed levee raise and improvements for McCorkle Creek (A/E report to Lexington Flood Control Coordinator)
- Contract Drawings for the 1980 Mount St. Helens Eruption levee raise work
- Contract Drawings for the 1982 Cowlitz River Levee Improvement for the Lexington Levee
- Report that included Lexington Levee in the "Cowlitz-Coweman Levee Final Report" completed by AE contractor Ogden Beeman & Associates in November 1985 for proposed levee improvements to meet 100-, 500-, and 500-year plus 4 feet protection

D.6.1.4. Surveys

Surveys for the levee top profiles include the 1980 pre-construction survey, 1982 As-Built survey, 2001 survey, 2007 National Levee Inventory survey (vertical accuracy of approximately 2 cm), and the 2007 LIDAR survey. There are cross sections from both the 2001 and 2007 surveys at selected locations.

D.6.1.5. Exploration Programs

There is limited subsurface information for the Lexington levee. The only explorations involved 4 borings with in-place SPT tests and 5 CPT (Cone Penetrometer Test) probes during the 2001 program at Lexington, and a few field permeability tests. Soil properties for the embankment and foundation soils were estimated from the 2001 explorations.

D.6.2. Lexington Levee Reaches

Figure 3.1 shows the division of the Lexington Levee into 5 reaches, LX1 – LX5. Figure 3.2 is a profile of the levee top and safe water level. Table 3.1 shows the station endpoints of each reach.

Table 3.1. Lexington Levee reaches

Reach	Station (2007 levee inventory station)		
LX1	0+00	to	20+44
LX2	20+44	to	88+00
LX3	88+00	to	93+50
LX4	93+50	to	112+00
LX5	112+00	to	End

D.6.2.1. Reach LX1

Reach LX1 is the upstream end of the system west of Westside Highway. The photo below shows the stockpile of material available for closing the highway.

Photo 3.1. Lexington reach LX1

D.6.2.2.Reach LX2

Reach LX2 extends from approximately station 20+44 to 88+00. This reach includes McCorkle Creek pump station.

Photo 3.2. Lexington reach LX2

D.6.2.3.Reach LX3

Reach LX3 extends from approximately station 88+00 to 93+50. There is dredged material on the landward side of the levee up to the crest elevation. The County reported that some of the dredged sand has been removed. This reach should be re-evaluated if significant volumes of sand are removed in the future, such that the levee has a distinct back-slope again.

Photo 3.3. Lexington reach LX3

D.6.2.4.Reach LX4

Reach LX4 extends from approximately station 93+50 to 112+00. The low spot in this reach is adjacent to an A-frame house.

Photo 3.4 a. Lexington reach LX4

Photo 3.4 b. Lexington reach LX4, low spot in front of A-frame house

D.6.2.5.Reach LX5

Reach LX5 extends from approximately station 112+00 to the downstream end of the system. Dredged material is against the landward slope of the levee in this reach.

D.6.3. Levee Reaches Screening

The critical part of reach LX1 is the Westside Hwy crossing. Located at the upstream end of the system, a high enough river stage could flood the community if the low spot is not raised. However, no further analysis was done for this location since the Lexington District is aware of the situation and there is a stockpile of material available to build an emergency embankment across the road.

As with the Castle Rock levee, the riverward part of the 1980 Lexington levee was also constructed on a sediment deposit resulting from the eruption of Mount St. Helens. The deposit at Lexington is different from the deposit at Castle Rock: the deposit at Lexington has much more silt so it is not as permeable as the deposit at Castle Rock. The deposit can be seen in boring log LX-DH-03, for example. It is the silty sand / sandy silt with low SPT blow counts. The 1980 deposit at Lexington is not as troublesome to seepage due to its lower permeability. However, to provide bearing capacity for construction equipment, a layer of granular fill was placed over the deposit. See Drawing CZG-5-2/5. The granular fill shows up in boring log LX-DH-02: it is the layer of gravel and cobbles up to 6 inches in size at a depth from 23.5 to 25 ft. This granular fill is significant to seepage. Seepage and stability analyses were performed for reaches LX2 and LX4.

No analyses were done for reaches LX3 and LX5 as dredged material is against the landward slope of the levee in these reaches.

D.6.4. Lexington Levee Seepage and Stability Analyses

Seepage and stability analyses were performed for the levee at stations 47+20 (reach LX2) and 105+00 (reach LX4).

D.6.4.1.Station 47+20 (reach LX2)

This levee section, by the apartment buildings northwest of Riverside Park, was selected for analysis due to its steep landward slope of 1.5:1. Figures 3.3 and 3.4 show the results of the seepage and stability analyses for the levee at station 47+20. Table 3.2 shows the soil properties used in the analyses. With the river at the 1980 DWS, the seepage exit gradient at the toe is 0.31 and the landward slope factor of safety against sliding is 1.9.

Table 3.2. Reach LX2, station 47+20 soil properties

	Quarry waste	Levee sand with silt	Granular fill	Drain rock	Foundation sand with silt / silt with sand	Foundation sand with gravel
Permeability, k (ft/min)	0.1	0.002	0.1	1	0.00004	0.004
Horiz. k / Vert. k	4	4	2	1	4	4
Unit weight (pcf)	135	140	125	110	120	130
c' (psf)	0	0	0	0	0	0
ϕ' (degrees)	40	40	37	30	30	37

D.6.4.2.Station 105+00 (reach LX4)

This levee section, located just upstream of the A-frame house, was selected for analysis due to its steep landward slope of 3:1. Figures 3.5 and 3.6 show the results of the seepage and stability analyses for the levee at station 105+00. Table 3.3 shows the soil properties used in the analyses. With the river at the 1980 DWS, the seepage exit gradient at the toe is 0.56 and the landward slope factor of safety against sliding is 2.1.

Table 3.3. Reach LX4, station 105+00 soil properties

	Quarry waste	Levee sand	Granular fill	Foundation silty sand / sandy silt	Foundation sand with silt
Permeability, k (ft/min)	0.1	0.02	0.1	0.00004	0.002
Horiz. k / Vert. k	4	4	2	4	4
Unit weight (pcf)	135	140	125	120	130
c' (psf)	0	0	0	0	0
ϕ' (degrees)	40	40	37	30	37

D.6.5. Discussion

The potential influence of the granular fill layer on seepage conditions can be seen in Figures 3.3 and 3.5. Consider Figure 3.5 for station 105+00. If the granular fill layer is as permeable as modeled, then only about 1/5 of the total head drop across the levee occurs through the granular fill. The granular fill allows high seepage pressures to penetrate about halfway beneath the levee, resulting in a relatively high hydraulic gradient beneath the landward half of the levee.

While the computed exit gradient, 0.31, at station 47+20 (reach LX2) is less than the computed gradient at station 105+00 (reach LX4), 0.56, which is approximately the safe value of 0.5, the reported presence of boils in reach LX2 during the 1996 flood event tempers any inclination to raise the SWL in this reach above the 1980 DWS. The 1996 flood stage was below the 1980 DWS. Figure 3.7 shows the boil locations reported by the County.

The conclusion of the seepage and stability analyses, with consideration of reported boils in 1996, is that the 1980 DWS should be used for the SWL for reaches LX2 and LX4.

D.6.6. Lexington Index Points

The Lexington levee has two index points, designated LeXington Index Point (LXIP) 1 and LXIP 2. It is assumed that the known low spots at McCorkle Creek pump station and the A-frame house can be raised temporarily in a flood event to match the top elevations of the adjacent levee sections. LXIP 1 has one of the lower top elevations in reach LX2. LXIP 2 was chosen in reach LX3 in order to determine the level of protection for an overtopping condition at Lexington.

Table 3.4. Lexington Index Points

Index Point	Station (2007 survey stationing)	Elevation (NAVD88) at which:	
		Probability of failure = 0	Probability of failure = 1
LXIP1*	70+00	38.2	45.7
LXIP2	88+83	42.6	42.6

POST-REPORT COMMENTS:
*** See Addendum for updated fragility curve points.**

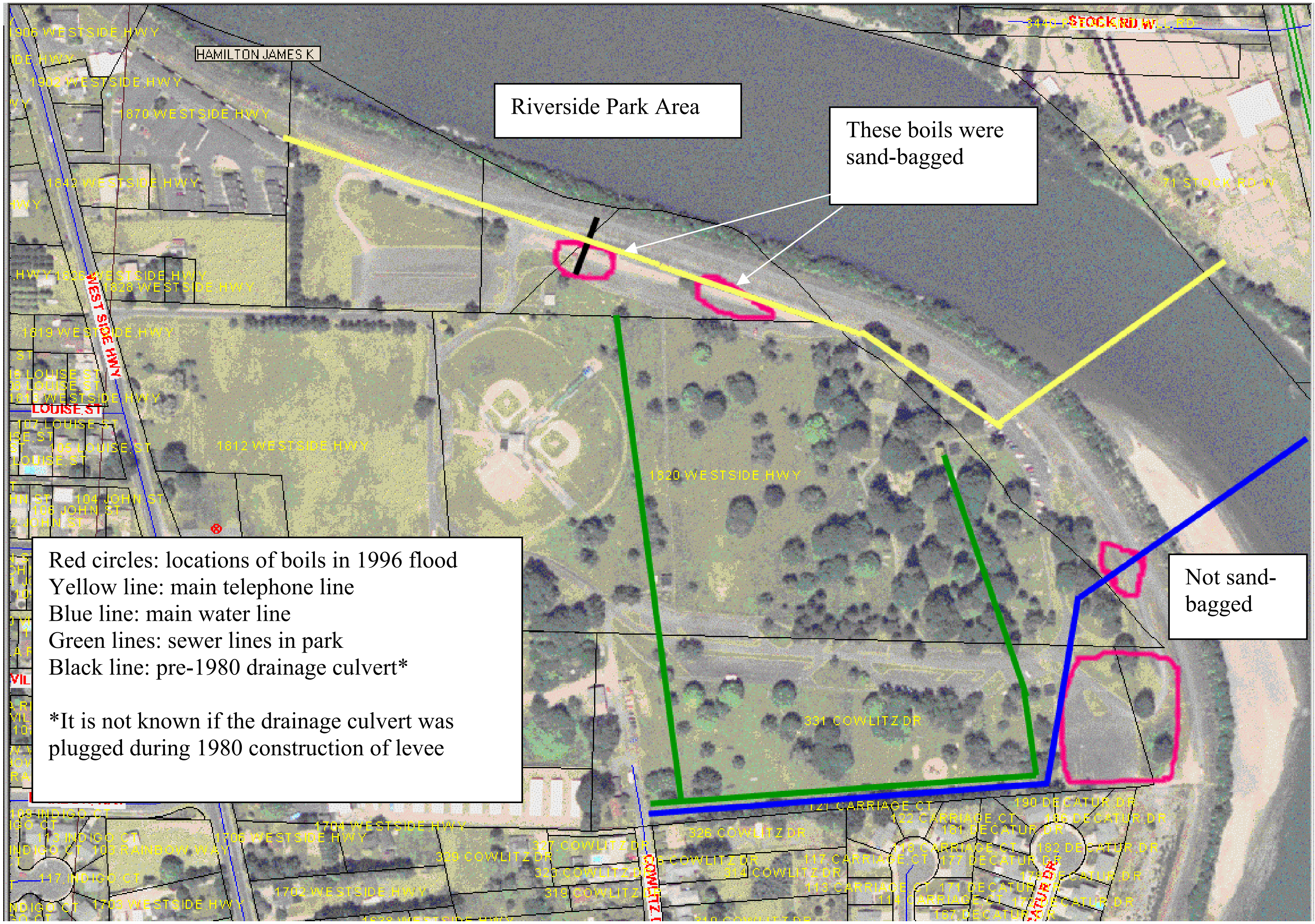


Figure 3.7. Locations of boils at Lexington levee during 1996 event, reported by Cowlitz County

D.7. LONGVIEW LEVEE

D.7.1. Information Reviewed

D.7.1.1. Construction History

Prior to 1923 the Longview area consisted of several levee districts that were not adequately protected on both the Cowlitz and Columbia River segments. The present levee district, Cowlitz County Consolidated Diking Improvement District No. 1 (CDID No. 1), also known as the Longview Levee, was formed and comprehensive levee and interior drainage works were constructed beginning in 1923. The design flood used was the 1894 flood level plus 5 feet of freeboard. The Cowlitz levee is composed of sand and gravel (hydraulic-fill), placed impervious clays, and mixed silts, sands, and clays (dragline and bucket). In 1935 and 1936 eroded areas along the Columbia were repaired by cost share. The Corps rehabilitated about 4 miles of levee on both the Cowlitz and the Columbia during 1939, placed stone protection, installed a pumping plant, and excavated 2,600 lineal feet of interior drainage canal. From the 1948 high water, seepage-caused damage was repaired by the Corps through emergency funding (State and Federal) for placement of an impervious clay blanket on the Cowlitz River side of the levee and gravel blankets in two areas on the landward side. The Columbia levee segment also required repairs. Design for improvements under the authorized 1950 Flood Control Act was not implemented by the Corps. However, the District had completed much of the improvements through their own efforts by 1978.

After the Mount St. Helens Eruption, there were several levee raises along the Cowlitz River beginning in 1980 that included embankment and rock protection, a new stop log structure, and temporary structures.

D.7.1.2. Inspection History

Inspections of the levee began in 1974 and have been continuous to the present. The levee remains active in the rehabilitation and inspection program.

D.7.1.3. Reports and Drawings

The following reports and drawings are available:

- Letter Report on Authorized Projects, Lower Columbia River and Tributaries, Volume I and II, 2 June 1952
- 1972 Report On Drainage Facilities And Requirement (A/E report)
- Contract Drawings for the 1980 Mount St. Helens Eruption levee raise work
- Contract Drawings for the 1982 Cowlitz River Levee Improvement for the Lonview Levee
- Report that included CDID No. 1 in the “Cowlitz-Coweman Levee Final Report” completed by AE contractor Ogden Beeman & Associates in November 1985 for proposed levee improvements to meet 100-, 500-, and 500-year plus 4 feet protection

D.7.1.4. Surveys

Surveys for the levee top profiles include the 1980 pre-construction survey, 1982 As-Built survey, 2001 survey, 2007 National Levee Inventory survey (vertical accuracy of approximately 2 cm), and the 2007 LIDAR survey. There are cross sections from the 2001 and 2007 surveys at selected locations.

D.7.1.5. Exploration Programs

There is limited subsurface information for the Longview levee. There were explorations completed during the early 1950's along the entire alignment for proposed improvements using both auger and drill holes. Recent explorations along the Cowlitz River segment involved 3 borings with in-place SPT tests and 5 CPT (Cone Penetrometer Test) probes during the 2001 program, and a few field permeability tests. Soil properties for the embankment and foundation soils were estimated from the 2001 explorations.

D.7.2. Longview Levee Reaches

Figure 4.1 shows the division of the Longview Levee into 6 reaches, LV1 – LV6. Figure 4.2 is a profile of the levee top and safe water level. Table 4.1 shows the station endpoints of each reach.

Table 4.1. Longview Levee reaches

Reach	Station (2007 levee inventory station)		
LV1	0+00	to	41+20
LV2	41+20	to	57+00
LV3	57+00	to	94+00
LV4	94+00	to	102+00
LV5	102+00	to	161+00
LV6	161+00	to	End – RM 2

D.7.2.1.Reach LV1

Reach LV1 extends from the upstream end of the levee to approximately station 41+20 at the Allen Street Bridge.

Photo 4.1. Longview reach LV1

D.7.2.2. Reach LV2

Reach LV2 extends from approximately station 41+20 to 57+00. According to survey data this is a low area in the system. The Hall of Justice building is shown in the photo below.

Photo 4.2. Longview reach LV2

D.7.2.3.Reach LV3

Reach LV3 extends from approximately station 57+00 to 94+00. Figure 4.2 shows spots where the top of levee is below the 1980 Design Water Surface.

Photo 4.3. Longview reach LV3

D.7.2.4.Reach LV4

Reach LV4 extends from approximately station 94+00 to 102+00. A concrete block wall has been built along this stretch on the riverward shoulder of the levee.

Photo 4.4. Longview reach LV4

D.7.2.5.Reach LV5

Reach LV5 extends from approximately station 102+00 to 161+00. Survey data indicates one low spot near the north end of Marine View Drive. Photo 4.5 b below shows where a slaughter house was removed and the landward slope is steeper than desirable.

Photo 4.5 a. Longview reach LV5

Photo 4.5 b. Longview reach LV5, removed slaughter house location

D.7.2.6.Reach LV6

Reach LV6 extends from approximately station 161+00 to the end of the levee. This reach is essentially high ground as dredged material has been placed on the landward side.

Photo 4.6. Longview reach LV6

D.7.3. Levee Reaches Screening

The levee in reach LV1 is shorter in height – i.e. the interior ground elevation is greater – than most of the levee reaches to the south. Low spots exist at road, driveway, and railroad crossings. Since the low spots are short in length, it is assumed that the levee district can plug the low spots in a flood event. No further analysis was performed for this reach.

Reach LV2 is the largest low spot in the levee system. The diking district has demonstrated that this reach can be raised to the 1980 DWS before a flood event. In 1995 the reach was raised in 8 hours. An index point will be established in this reach for overtopping.

Most of the levee in reaches LV3 – LV5 has a suitable backslope ranging from 5H:1V to 8H:1V. The exception is the short stretch in reach LV3 in the vicinity of station 60 where 1st Avenue (411) is closer to the levee. Seepage and stability analyses were performed for the levee at station 61+50, where the landward slope is 3H:1V.

No further analysis was done for reach LV4. Even though the survey data shows the levee behind the block wall to be slightly (less than 6 inches) below the 1980 Design Water Surface, the presence of the block wall validates the Design Water Surface.

In reach LV5, an index point was established at station 119+00 for overtopping. The levee section where the slaughter house was removed (station 149 vicinity) should be rehabilitated to match the adjacent levee section.

There is a dredge disposal site landward of the levee in reach LV6. No further analysis is required for this reach.

D.7.4. Longview Levee Seepage and Stability Analyses

Seepage and stability analyses were performed for the levee at station 61+50 (reach LV3).

D.7.4.1.Station 61+50 (reach LV3)

This levee section was selected for analysis due to its steep landward slope, compared to the rest of the levee, of 3:1. Figures 4.3 and 4.4 show the results of the seepage and stability analyses for the levee at station 61+50. Table 4.2 shows the soil properties used in the analyses. With the river at the 1980 DWS, the seepage exit gradient at the toe, between the levee and the road, is 0.25 and the landward slope factor of safety against sliding is 1.8.

Table 4.2. Reach LV3, station 61+50 soil properties

	Levee sand with gravel, some silt	Foundation silty sand	Road base
Permeability, k (ft/min)	0.002	0.0002	0.2
Horiz. k / Vert. k	4	4	4
Unit weight (pcf)	122	120	135
c’ (psf)	0	0	0
ϕ’ (degrees)	37	35	40

D.7.4.2.Discussion

Since the exit gradient and landward slope factor of safety against sliding are adequate for the levee section with the steepest backslope (station 61+50), the SWL is the 1980 DWS.

D.7.5. Longview Index Points

The Longview levee has four index points, designated LongView Index Point (LVIP) 1 – LVIP 4. LVIP 1 and LVIP 3 are overtopping index points. LVIP 4 is not a low spot, but is within a stretch of levee where the top elevation is relatively low; see Figure 4.2.

Table 4.3. Longview Index Points

Index Point	Station (2007 survey stationing)	Elevation (NAVD88) at which:	
		Probability of failure = 0	Probability of failure = 1
LVIP1	50+75	35.1	35.1
LVIP2	61+50	34.8	37.4
LVIP3	119+00	32.8	32.8
LVIP4	140+30	32.0	32.5

D.8. KELSO LEVEE

D.8.1. Information Reviewed

D.8.1.1. Construction History

The lower Kelso levee is part of the Cowlitz County Consolidated Diking Improvement District No. 3 (formerly No. 13 and 2 prior to consolidation). The upper Kelso levee is the Cowlitz County Drainage Improvement District No. 1. The lower Kelso diking district along the Cowlitz River was organized in 1918 and constructed a levee 2 feet above the 1876 flood level at the downstream end and 5 feet above at the upstream end. The December 1933 flood destroyed much of the levee and it was restored by the Civil Works Administration (CWA). 1936 construction work by the CWA, supervised by the Corps, improved the levee to 3 feet above the 1933 flood level. In 1938 the Corps placed riprap on the levee slope. Seepage from the 1948 indicated improvements were required and this was completed by the Corps in 1956 by placement of pervious materials on the landward side of the levee embankment.

After the 1980 eruption, the levee raise by the Corps added embankment material and slope protection or riprap to 2 feet above the design level of protection to the Kelso levee. The upper Kelso levee saw a Corps designed and constructed levee raise immediately adjacent to the railroad embankment. Levee embankment and riprap rock was constructed to 2 feet above design flood level and completed in 1981. Additional work for an emergency temporary levee raise by the Corps in 1982 and 1983 saw reinforcement and improvements on both the upper and lower Kelso levee that varied based on levee location. Some temporary measures were later removed when no longer required.

D.8.1.2. Inspection History

Annual inspections by the Corps for both Drainage Improvement District No. 1 and Consolidated Diking Improvement District No. 3 began in 1974 and were continuous through 2007. The levees are both active in the rehabilitation and inspection program.

D.8.1.3. Reports and Drawings

The following reports and drawings are available:

- Letter Report on Authorized Projects, Lower Columbia River and Tributaries, Volume I and II, 2 June 1952
- Design Memorandums Cowlitz Co. CDID No. 2, Cowlitz Co. CDIC No. 13, October 1962
- 1972 Report On Drainage Facilities And Requirement (A/E report)
- Contract Drawings for the 1980 Mount St. Helens Eruption levee raise work
- Contract Drawings for the 1982 Cowlitz River Levee Improvement for the upper and lower Kelso Levee
- Report that included upper and lower Kelso Levee in the "Cowlitz-Coweman Levee Final Report" completed by AE contractor Ogden Beeman & Associates in November 1985 for proposed levee improvements to meet 100-, 500-, and 500-year plus 4 feet protection
- Kelso Levee Improvement Design Memorandum No. 14, November 1987

D.8.1.4. Surveys

Surveys for the levee top profiles include the 1980 pre-construction survey, 1982 As-Built survey, 2001 survey, 2007 National Levee Inventory survey (vertical accuracy of approximately 2 cm), and the 2007 LIDAR survey. There are cross sections from both the 2001 and 2007 surveys at selected locations.

D.8.1.5. Exploration Programs

There is limited subsurface information for the Kelso levee. There were explorations completed during the early 1950's along the entire alignment for proposed improvements using both auger and drill holes. Recent explorations along the Cowlitz River segment involved 3 borings with in-place SPT tests and 5 CPT (Cone Penetrometer Test) probes during the 2001 program, and a few field permeability tests. Soil properties for the embankment and foundation soils were estimated from the 2001 explorations.

D.8.2. Kelso Levee Reaches

Figure 5.1 shows the division of the Kelso Levee into 10 reaches, KL1 – KL10. Figure 5.2 is a profile of the levee top and safe water level. Table 5.1 shows the station endpoints of each reach.

The upstream end of the Kelso levee system is a railroad tunnel that may require closure in a flood event. It is assumed that the drainage district will be able to close the tunnel if necessary using the Corps-provided concrete structures located at the tunnel entrance, earth or rock fill, sandbags, or a combination of these items.

Table 5.1. Kelso Levee reaches

Reach	Station (2007 levee inventory station)		
N. Kelso			
KL1	0+00	to	31+00
KL2	31+00	to	112+86
S. Kelso*			
KL3	112+86	to	115+93
KL4	115+93	to	141+70
KL5	141+70	to	143+45
KL6	143+45	to	158+69
KL7	158+69	to	178+75
KL8	178+75	to	191+70
KL9	191+70	to	255+73
KL10	255+73	to	End – RM 2

* Reach KL2 covers the upper part of S. Kelso, too

D.8.2.1.Reach KL1

Kelso reach KL1 extends from station 0+00 to approximately station 31+00. The reach is composed of—from upstream to downstream—a railroad embankment, a steel sheet pile wall, a short embankment, a road crossing, and a concrete flood wall.

Kelso Levee Improvement Design Memorandum No. 14 (1987) concluded that no change was needed for the sheet pile wall to meet the 143-yr level of protection. Drawing CZR-7-5/3 shows the improvement made to the short embankment (located south of the sheet pile wall) to provide the 143-yr level of protection.

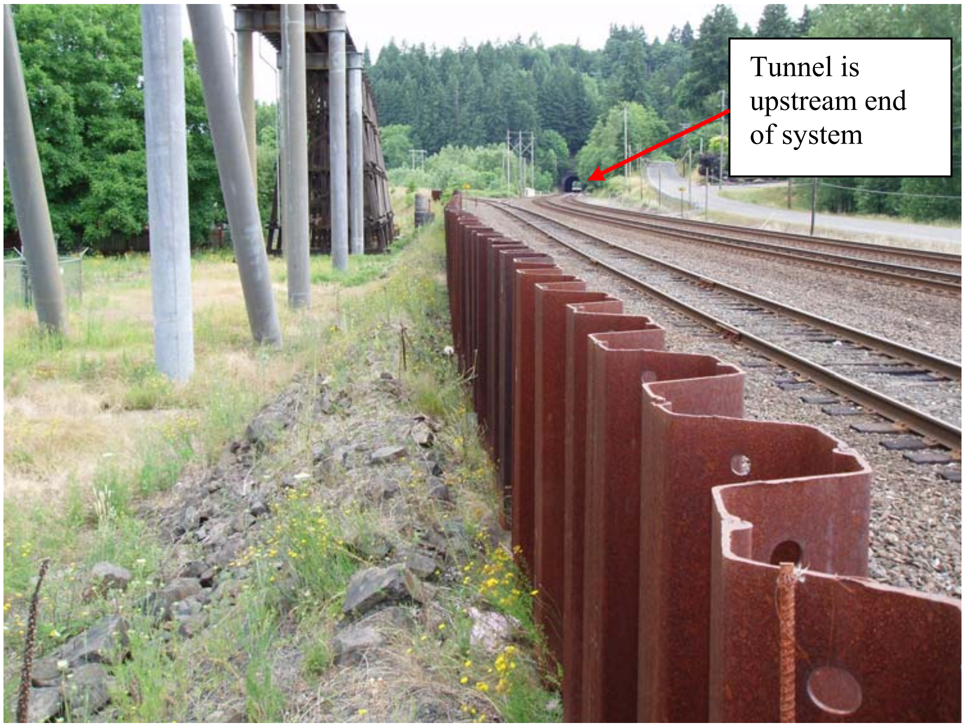


Photo 5.1 a. Kelso reach KL1, sheet pile wall

Photo 5.1 b. Kelso reach KL1, short embankment

Photo 5.1 c. Kelso reach KL1, road crossing

Photo 5.1 d. Kelso reach KL1, concrete flood wall

D.8.2.2.Reach KL2

Reach KL2 extends from approximately station 31+00 to 112+86. The levee section has railroad tracks close to the landside toe as shown in the photo below.

Photo 5.2. Kelso reach KL2

D.8.2.3.Reach KL3

Reach KL3 is Rotary Landing beneath the Allen Street bridge. This is an approximately 300-ft-long low spot in the levee.

Photo 5.3. Kelso reach KL3

D.8.2.4. Reach KL4

Reach KL4 extends from approximately station 115+93 to 141+70. The levee section has railroad tracks close to the landside toe as shown in the photo below.

Photo 5.4. Kelso reach KL4

D.8.2.5.Reach KL5

Reach KL5 is the Mill Street road crossing.

Photo 5.5. Kelso reach KL5

D.8.2.6.Reach KL6

Reach KL6 extends from approximately station 143+45 to 158+69. The levee section has railroad tracks close to the landside toe as shown in the photo below.

Photo 5.6. Kelso reach KL6

D.8.2.7.Reach KL7

Reach KL7 extends from approximately station 158+69 to 178+75: South River Road to Olive Street. The levee section in this reach is wider than in reaches KL2, KL4, and KL6.

Photo 5.7. Kelso reach KL7

D.8.2.8.Reach KL8

Reach KL8 extends from approximately station 178+75 to 191+70. Dredged sand previously placed riverward of the levee has been removed along this stretch of levee. Though the presence of riprap on the exposed riverward slope is not apparent in the photos below, riprap is on the levee slope below a thin layer of remaining dredged sand. The presence of the riprap, the placement of which is shown on Drawings CZR-7-5/5 and /6, was verified by a field investigation. Photo 5.8 c shows South River Road and the relatively steep landward slope in this reach.

Photo 5.8 a. Kelso reach KL8

Photo 5.8 b. Kelso reach KL8

Photo 5.8 c. Kelso reach KL8

Photo 5.8 d. Kelso reach KL8

D.8.2.9.Reach KL9

Reach KL9, which extends from approximately station 191+70 to 255+73, is essentially high ground now due to placement of dredged material adjacent to the levee.

Photo 5.9 a. Kelso reach KL9

Photo 5.9 b. Kelso reach KL9

D.8.2.10. Reach KL10

Reach KL10 extends from approximately station 255+73 to the downstream end of the levee: the mouth of the Coweeman River. This reach is a broad sand railroad embankment. The railroad embankment is about 27 ft wide at the top. A 24-ft-wide side-road berm is located on the landward side of the railroad embankment, with a top elevation about 6 ft below the top of the railroad embankment.

Photo 5.10. Kelso reach KL10

D.8.3. Levee Reaches Screening

The critical areas in reach KL1 are the low spots upstream of the sheet pile wall and at the road crossing. The low spot upstream of the sheet pile wall will be used as an index point for overtopping.

Reaches KL2 – KL6 are similar. Among reaches KL2, KL4, and KL6, station 60+53 in reach KL2 was selected for further analysis due to its critical combination of steepness and height of landward slope. Rotary Landing (KL3) and Mill Street crossing (KL5) are low spots that can be raised during a flood event (e.g. with sandbags) to match the top elevation and level of protection of the adjacent levee sections.

Reach KL7 was not analyzed further because the levee section in this reach is wider than in reaches KL2, KL4, and KL6. However, an index point was established because the top elevation of the levee is relatively lower in this reach.

Reach KL8 was chosen for further analysis due to its steep (2.6:1) and high landward slope, which is now important since the dredged material has been removed from the riverward side of the levee.

No further analysis is required for reach KL9, as this reach is essentially high ground. Reach KL10 was not analyzed further either as the embankment is very broad in this reach.

D.8.4. Kelso Levee Seepage and Stability Analyses

Seepage and stability analyses were performed for the levee at stations 60+53 (reach KL2) and 186+76 (reach KL8).

D.8.4.1. Station 60+53 (reach KL2)

This levee section was selected for analysis due to its critical combination of steepness and height of landward slope. Figures 5.3 and 5.4 show the results of the seepage and stability analyses for the levee at station 60+53. Table 5.2 shows the soil properties used in the analyses. With the river at the 1980 DWS, the seepage exit gradient at the toe is 0.32 and the landward slope factor of safety against sliding is 2.0.

Table 5.2. Reach KL2, station 60+53 soil properties

	Levee sand with gravel	1980 Foundation sand	Foundation sand with gravel
Permeability, k (ft/min)	0.02	0.02	0.0002
Horiz. k / Vert. k	4	4	4
Unit weight (pcf)	122	120	125
c' (psf)	0	0	0
ϕ' (degrees)	36	34	37

D.8.4.2. Station 186+76 (reach KL8)

This levee section was selected for analysis due to its relatively steep and high landward slope. Figures 5.5 and 5.6 show the results of the seepage and stability analyses for the levee at station 186+76. Table 5.3 shows the soil properties used in the analyses. With the river at elevation 30.4 ft, which is below the 1980 DWS, the upward seepage exit gradient at the toe is 0.41 and the landward slope factor of safety against sliding is 1.5. The seepage analysis indicates that seepage will emerge from the bottom 3 ft of the slope. A river stage of 30.4 ft will be taken as the SWL for this levee section, as discussed further below.

Table 5.3. Reach KL8, station 186+76 soil properties

	Levee sand with gravel	Road base	Sand	Foundation sand with gravel
Permeability, k (ft/min)	0.02	0.2	0.02	0.0002
Horiz. k / Vert. k	4	4	4	4
Unit weight	122	135	120	125

(pcf)				
c' (psf)	0	0	0	0
ϕ' (degrees)	36	40	34	37

D.8.4.3.Discussion

Based on the analyses at station 60+53, the 1980 DWS is considered the SWL for most of the levee. The levee in reach KL8 is the exception. The lower SWL for this reach is shown in Figure 5.2. The reason for the lower SWL here is the emergence of seepage on the steep landward slope. This seepage can cause sloughing of the cohesionless levee sand. Seepage emerging from the bottom 3 ft of the slope is considered manageable because the slope can be buttressed (e.g. with crushed rock or quarry waste) if sloughing occurs, as South River Road runs along the toe of the levee, providing easy access for trucks and equipment.

D.8.5. Kelso Index Points

The Kelso levee has four index points, designated KeLso Index Point (KLIP) 1 – KLIP 4. KLIP 1 is the low spot upstream of the sheet pile wall. KLIPs 2 – 4 are representative of reaches KL2, KL7, and KL8.

If in the future dredged material is removed from against the levee in reach KL9, then this reach will require re-evaluation. The level of protection may decrease if dredged material is removed, as it did for reach KL8.

Table 5.4. Kelso Index Points

Index Point	Station (2007 survey stationing)	Elevation (NAVD88) at which:	
		Probability of failure = 0	Probability of failure = 1
N. Kelso			
KLIP1	15+26	37.7	37.7
KLIP2	57+13	37.4	40.3
S. Kelso			
KLIP3	171+86	33.5	34.5
KLIP4	186+76	30.4	33.4

D.9. COWEEMAN LEVEE

D.9.1. Information Reviewed

D.9.1.1. Construction History

The original Coweman levee district was organized in 1913 and constructed almost 4 miles of levee that had 1-foot of freeboard above the 1876 flood level. This also included tide gates and culverts for drainage. In 1926 some repairs were made to the levee and a pumping plant was constructed. The flood of June 1933 caused failure of the levee at the pumping plant and inundation of the district. A State of Washington grant allowed repair of the levee and replacement of the pump house with two pumps. The flood of December 1933 caused breaches and overtopping at the upper end of the levee that again inundated the district. Repairs were made of the damaged levee. In 1935 and 1936, the Corps supervised a federal Works Progress Administration project for reconstruction of the levee with local cost share participation. This included a crest raise resulting in a 3-foot freeboard over the 1894 flood level and riprap protection on the riverward slope. Two tideboxes were constructed for the project. During the 1948 flood, inundation was prevented when emergency flood fights kept high seepage from failing the railroad embankment into an adjacent borrow pit. In 1949 the borrow pit was backfilled by the Corps in the area adjacent to the Coweman levee and the railroad embankment was buttressed with a sand fill that reached a height equal to the 1894 flood level. In 1956 fill was placed on the landward slope and a drainage ditch was filled. In 1965 improvements were made to the levee and pump station.

After the 1980 Mount St. Helens Eruption, no permanent levee raise was made to the Coweman levee.

D.9.1.2. Inspection History

Annual inspections by the Corps on the Consolidated Diking Improvement District No. 3 levee began in 1974 and were continuous through 2007. The levee remains active in the rehabilitation and inspection program.

D.9.1.3. Reports and Drawings

The following reports and drawings are available:

- Letter Report on Authorized Projects, Lower Columbia River and Tributaries, Volume I and II, 2 June 1952, that included proposed improvements for CDID No. 3 (Coweman)
- Design Memorandums Cowlitz Co. CDID No. 2, Cowlitz Co. CDIC No. 13, October 1962
- Report that included the Coweman Levee in the “Cowlitz-Coweman Levee Final Report” completed by AE contractor Ogden Beeman & Associates in November 1985 for proposed levee improvements to meet 100-, 500-, and 500-year plus 4 feet protection

D.9.1.4. Surveys

Surveys for the levee top profiles include the 1980 pre-construction survey, 1982 As-Built survey, 2001 survey, 2007 National Levee Inventory survey (vertical accuracy of approximately 2 cm), and the 2007 LIDAR survey. There are cross sections from both the 2001 and 2007 surveys at selected locations.

D.9.1.5.Explorations Programs

There is limited subsurface information for the Coweman levee. There were explorations completed during the early 1950's along the entire alignment for proposed improvements using both auger and drill holes. Recent explorations along the Cowlitz River segment involved 2 borings with in-place SPT tests and 5 CPT (Cone Penetrometer Test) probes during the 2001 program, and a few field permeability tests. Soil properties for the embankment and foundation soils were estimated from the 2001 explorations.

D.9.2. Coweeman Levee Reaches

Figure 6.1 shows a plan view of the Coweeman Levee. Figure 6.2 is a profile of the levee top and safe water level.

Due to its uniformity, the Coweeman levee was not divided into reaches. There is a dredge disposal site near the mouth of the river that creates high ground adjacent to the levee. This area would be treated differently if a levee improvement were to occur. The high ground area is not a problematic area and is not represented by an index point.

The photos below show the Coweeman levee.

Photo 6.1. Coweeman, riverward side near upstream end of levee

Photo 6.2. Coweeman, landward side near upstream end of levee

Photo 6.3. Coweeman, riverward side of levee in vicinity of station 65 – 70

Photo 6.4. Coweeman, landward side of levee in vicinity of station 65 – 70

Photo 6.5. Coweeman, levee at I-5 crossing at station 137

D.9.3. Coweeman Levee Seepage and Stability Analyses

The Coweeman levee foundation is considerably different from that of the other four levees. The foundation consists of a relatively impervious “blanket” of plastic silts and lean clays underlain by stratified silty sands and sands. The blanket thickness ranges from 4 to 18 ft and the depth of the lower silty sands and sands is unknown. This condition of a relatively thin landside blanket over a thick, pervious foundation is often problematic in terms of underseepage, as high seepage pressures tend to build beneath the blanket, causing a large hydraulic gradient across the blanket and the potential for uplift or erosion and piping.

Figures 6.3 and 6.4 show the results of seepage and stability analyses for the levee at station 29+80. This levee section has the highest ratio of height to width among the surveyed cross sections. Table 6.1 shows the soil properties used in the analyses. With the river at elevation 25.0 ft (see why below), the hydraulic gradient across the blanket is 0.5 and the landward slope factor of safety against sliding is 1.8.

The 1962 DWS is elevation 23.8 ft for most of the levee (increasing slightly at the upstream end). During the 1996 event, the peak stage in the Cowlitz River at the confluence with the Coweeman was approximately 25.0 ft. Several boils occurred when the Coweeman River was near its peak in 1996. The boils were successfully circled with sandbags. Given the diking district’s demonstrated ability to monitor and flood-fight during the 1996 event, the satisfactory performance of the levee during the event with standard remedial work, and the favorable results of the seepage and stability analyses with the river at elevation 25.0 ft, the SWL for the Coweeman River is 25.0 ft.

Table 6.1. Coweeman station 29+80 soil properties

	Levee silty sand	Levee sand	Foundation blanket	Foundation silty sand
Permeability, k (ft/min)	0.0002	0.02	0.00002	0.0002
Horiz. k / Vert. k	4	4	4	4
Unit weight (pcf)	120	120	110	120
c' (psf)	0	0	0	0
ϕ' (degrees)	35	35	32	37

6.4 Coweeman Index Points

The Coweeman levee has two index points, designated CoWeeman Index Point (CWIP) 1 and CWIP 2. CWIP 1 is the low point in the upstream portion of the levee and CWIP 2 is the low point in the downstream portion of the levee. It is assumed that the low points at the bridge crossings can be temporarily raised, as was done at Grade Street during the January 2009 event. See photo below.

Table 6.2. Coweeman Index Points

Index Point	Station (2007 survey stationing)	Elevation (NAVD88) at which:	
		Probability of failure = 0	Probability of failure = 1
CWIP1*	47+74	25.0	28.3
CWIP2*	172+83	25.0	27.4

POST-REPORT COMMENTS:
*** See Addendum for updated fragility curve points.**

Photo 6.6. Coweeman: quarry waste wrapped in plastic sheeting across Grade Street

D.10. SUMMARY

SWL profiles are shown in Figures 2.2 (Castle Rock), 3.2 (Lexington), 4.2 (Longview), 5.2 (Kelso), and 6.2 (Coweeman). Index points for all the levees are shown in Table 7.1.

For the Castle Rock levee, the SWL is at or above the 1980 DWS upstream of the bridge, and above the 1956 DWS downstream of the bridge.

For the Lexington levee, the SWL is at or above the 1980 DWS.

For the Longview levee, the SWL is at the 1980 DWS.

For the Kelso levee, the SWL is the 1980 DWS except in two locations: upstream of the sheet pile wall in reach KL1, and reach KL8. The railroad grade is the SWL just upstream of the sheet pile wall in reach KL1. In reach KL8, the SWL is about 3 ft below the levee top. The reason for the lower SWL here is the predicted emergence of seepage on the steep landward slope.

For the Coweeman levee, the SWL is elevation 25.0 ft, which is approximately the peak stage at the mouth of the river in the 1996 flood event.

Table 7.1. Index Points for All Levees

Index Point	Station (2007 survey stationing)	Elevation (NAVD88) at which: Probability of failure = 0	Elevation (NAVD88) at which: Probability of failure = 1
CRIP1*	10+00	57.4	61.5
CRIP2	32+14	57.3	60.9
CRIP3**	63+00	55.5	58.5
LXIP1**	70+00	38.2	45.7
LXIP2	88+83	42.6	42.6
LVIP1	50+75	35.1	35.1
LVIP2	61+50	34.8	37.4
LVIP3	119+00	32.8	32.8
LVIP4	140+30	32.0	32.5
N. Kelso			
KLIP1	15+26	37.7	37.7
KLIP2	57+13	37.4	40.3
S. Kelso			
KLIP3	171+86	33.5	34.5
KLIP4	186+76	30.4	33.4
CWIP1**	47+74	25.0	28.3
CWIP2**	172+83	25.0	27.4

POST-REPORT COMMENTS:
*** Pf = 0 up to top of levee for this index point after installation of seepage cutoff wall in 2009.**
**** See Addendum for updated fragility curve points.**

Appendix E. Cowlitz River Hydrologic Cross Sections

E.1. INTRODUCTION

Between August 10th and August 19th, 2009, David Evans and Associates, Inc. (DEA) conducted 88 hydrologic cross-sections on the Cowlitz River, from Cowlitz River Miles 0.0 to 20.2, for the U.S. Army Corps of Engineers (USACE), Portland District. This task order contained an additional Only When Authorized task, containing 20 sections, from river mile 0.0 to 3.4 and was authorized prior to mobilization.

The statement of work specified data to be acquired to high water lines on both sides of the channel and over recent depositional features along the banks. Digital photographs were taken upstream, across stream and downstream at each end of the individual cross-section. Each photo was given a file name corresponding to the feature the photo is showing. This report describes the control used for the surveys, data acquisition methodology and data processing procedures. In addition to this report, deliverables include a project CD-ROM containing ASCII point data and digital photographs of individual cross-sections.

E.2. DATUMS AND PROJECT CONTROL

Conducting a survey on an established coordinate system enables the survey to be reproduced at a later date with repeatable results. For this survey, field operations were conducted on final data sets generated using the North American Datum of 1983 (NAD83) horizontal datum, projected to the State Plane Coordinate System (SPCS), Washington South Zone, with units in U.S. feet. The vertical datum used for this survey is the North American Vertical Datum of 1988 (NAVD-88). The accuracy requirements were 3 feet or less, in the horizontal positioning and 0.5 feet or less, in the vertical accuracy.

Positioning for the hydrographic survey vessel was provided by a differential global positioning system (DGPS), receiving differential corrections from the Fort Stevens, Oregon DGPS reference station maintained by the U.S. Coast Guard. The DGPS accuracy is a sub-meter positioning system and provides horizontal positioning only. A real-time kinematic (RTK) global positioning system (GPS) rover with data collector was used for water surface observations, acquisition of data not obtainable by the single beam echosounder and to provide horizontal position checks for the DGPS system on the hydrographic survey vessel. The RTK-GPS system also utilized Russia's Global Orbiting Navigation Satellite System (GLONASS), providing the RTK-GPS system with additional satellites to increase the usable acquisition time and system performance. The RTK-GPS system provides centimeter accuracy for horizontal and vertical positioning. The RTK-GPS system received corrections via cellular modem from the Washington State Reference Network (WSRN), which uses North American Datum 1983 (NAD83) and a network of Continuously Operating Reference Stations (CORS96) Epoch 2002.00. Geoid 2003 was used to obtain corrected orthometric elevations in NAVD-88 vertical datum.

The existing USACE survey control monuments “15.1R” and “GERHART-2” were used as position and elevation checks of the RTK-GPS system by the land surveyor. The horizontal position check

did not deviate more than 0.26 feet from published values. The vertical position check did not deviate more than 0.14 feet from published values. The standard deviation did not exceed 0.10 feet from the horizontal and vertical position checks performed.

Position checks were performed on a daily basis prior to the acquisition of the cross-sections to provide a horizontal position check for the survey vessel DGPS system. This was performed by acquiring an RTK-GPS position on a random non-monumented point accessible by the survey vessel's DGPS antenna. The results of the checks performed by the survey vessel's DGPS system are provided in Table 1, as differences of northing and easting, in relation to the RTKGPS derived coordinates.

Table E. 1. Survey Vessel DGPS Observations vs. Land Surveyors RTK-GPS Observations

	Difference (feet)	
Date	**Northing**	**Date**
8/10/2009	0.03	8/10/2009
8/11/2009	-0.12	8/11/2009
8/12/2009	-0.50	8/12/2009
8/13/2009	1.27	8/13/2009
8/14/2009	1.12	8/14/2009
8/17/2009	-1.13	8/17/2009
8/18/2009	-0.43	8/18/2009
8/19/2009	0.23	8/19/2009

E.3. BATHYMETRIC SURVEY METHODOLOGY

E.3.1. SURVEY COVERAGE

The project specifications required data acquisition, approximately perpendicular to river flow, along pre-determined river cross-sections provided by the USACE, Portland District. The coverage was to extend from the high water lines on both sides of the river including shoals, islands and areas of recent deposits.

There are three sections that have data gaps due to safety concerns encountered in the field during acquisition. These areas were too shallow for the survey vessel to acquire accurate bathymetric data, thus land surveying methods were attempted instead. Several attempts were initiated to acquire the data by the land surveyor, however conditions deemed to be hazardous, jeopardizing personal safety. Observations from the field crew determined that the data could be interpolated between the data points collected with a high degree of confidence. The section at river mile 3.00 has horizontal data gaps spanning 11 feet and 21 feet, and the section at river mile
3.50 has a horizontal data gap spanning 27 feet. These areas were not covered due to extremely soft bottom conditions (similar to quick sand or fluid mud) resulting in unsafe walking conditions and too shallow for vessel operations. As this is a fluid mud bottom, it is safe to assume a uniform bottom across these gaps. The section at river mile 19.10 has a horizontal data gap spanning 34 feet and was not collected due to the extremely swift water, making the land surveyor's crossing hazardous yet too shallow for vessel operations. Due to the shallow water, the bottom across this gap was observed to be uniform between bathymetric and topographic data points collected. It was observed that a linear interpolation between the data points would be accurate to better than 0.5 feet vertically and therefore meeting accuracy requirements. At theses sections the vessel operator determined a safe landing location, often several hundred feet from the cross-section. The land surveyor hiked back along the

shoreline to obtain upland topography and obtain submerged data points as far from the bank as was deemed safe. As mentioned above, this resulted in data gaps in a few locations.

E.3.2. SURVEY VESSEL AND CREW

The vessel for this survey was a 19-foot aluminum hulled jet sled, powered by a 90-horsepowered outboard jet engine, specifically designed for shallow river hydrographic surveys, owned and operated by DEA.

The hydrographic survey crew consisted of a senior hydrographer/professional land surveyor, a vessel operator/hydrographer and a land survey party chief from DEA. The crew has conducted numerous shallow water river surveys and has had extensive training in hydrographic surveys and land surveying.

E.3.3. POSITIONING AND NAVIGATION

Horizontal positioning for the single beam echosounder data was acquired with a Trimble DSM132 DGPS, receiving differential corrections from the Fort Stevens, Oregon DGPS reference station maintained by the U.S. Coast Guard. The DGPS accuracy is a sub-meter positioning system and provides horizontal positioning only. A Leica Smart Rover RTK global positioning system (GPS) with a Leica data collector was used for water surface observations, acquisition of data not obtainable by the single beam echosounder and to provide horizontal position checks for the DGPS system on the hydrographic survey vessel. The RTK-GPS system also utilized GLONASS providing the RTK-GPS system with additional satellites to increase the usable acquisition time and system performance. The RTK-GPS system provides centimeter accuracy for horizontal and vertical positioning. The RTK-GPS system received corrections via cellular modem from the WSRN, which uses NAD83 and a network of CORS. Geoid 2003 was used to obtain corrected orthometric heights relative to NAVD-88. Portions of the banks of the Cowlitz River contain a heavy tree canopy affecting the use of RTK-GPS for reliable positioning. A current GPS ephemeris was used on a daily basis for planning purposes, to provide use of the best GPS constellation times, maintain positional accuracy and to increase field efficiency.

DGPS position data was used in real-time to provide navigation information to the vessel operator and was time tagged and logged with single beam echosounder data. The actual cross-section alignment and survey vessel tracks are displayed with single beam coverage in real-time on a monitor located at the helm to aid in maintaining cross-section off-line specifications.

E.3.4. WATER SURFACE OBSERVATIONS

Water surface measurements were obtained by RTK-GPS using a Leica Smart Rover RTK-GPS total station system with a Leica data collector. The RTK-GPS system received corrections via cellular modem from the WSRN, which uses NAD83 and a network of CORS. Geoid 2003 was used in the Leica data collector to obtain corrected orthometric heights relative to NAVD-88. The water surface elevations were collected at the edge of water and time stamped on each individual cross-section. On a majority of the cross-sections, a water surface elevation was observed in real-time near the center of the river to verify the edge of water observations. This was implemented due to the heavy tree

canopy along the river bank. The water surface elevations were recorded in the hydrographic field logs and in the land surveyor's field notes. The edge-of-water elevations were time-stamped and are part of the deliverables. The water surface observations were entered into the Hypack Single Beam Editor during processing for each individual cross-section. All soundings and ground data collected for this survey are submitted as elevations in U.S. feet and referenced to NAVD-88.

E.3.5. SINGLE BEAM DATA ACQUISITION

Bathymetric data acquisition was collected using an integrated hydrographic data collection system installed on a 19-foot jet sled. The system was composed of a laptop computer running Hypack MAX software, interfaced with an Odom Echotrac CV-100 single beam sonar system for depth acquisition and a Trimble DSM-132 DGPS system for horizontal positioning.

Soundings were acquired with Odom Echotrac CV100 single beam bathymetric sonar using a frequency of 200 kilo hertz (kHz) and an Odom SM200/200, four-degree single beam transducer, specially designed for shallow water bathymetric acquisition. The manufacturer of the Odom Echotrac CV100 stated the instrument's accuracy of 0.10 feet +/- 0.1 % of the depth. The update rate from the Odom Echotrac CV100 was set at 15 hertz (Hz). The Odom Echotrac CV100 was interfaced to a laptop computer running Hypack software using a network cable and provided real-time depth acquisition. Hypack records the digital depth and full water column sonar echogram that is reviewed during post-processing.

The sound velocity profile of the water column was measured using an Odom Digibar Pro. The manufacturer stated accuracy of the Odom Digibar Pro is +/- 1 foot per second. The sound velocity of the water column was measured several times each day. The average sound velocity of each sound velocity profile was entered into the Odom Echotrac CV100 echosounder after each sound velocity cast.

The horizontal positioning for the single beam echosounder data was acquired with a Trimble DSM-132 DGPS, receiving differential corrections from the Fort Stevens, Oregon DGPS reference station, maintained by the U.S. Coast Guard. The DGPS accuracy is a sub-meter positioning system and provides horizontal positioning only. The Trimble DSM-132 DGPS was interfaced to Hypack with a serial computer cable with an update rate of 5 HZ.

E.4. EQUIPMENT CALIBRATION

E.4.1. LATENCY TEST

To confirm the latency (time delays) in the time tagging of the sensor data between the DGPS positioning system and the Odom Echotrac CV100 sonar, a latency test was conducted. The latency test lines are a pair of lines collected in opposite directions along the same alignment, over a prominent feature such as a slope. The data collected in the latency test lines is then analyzed in Hypack post processing software to determine the latency. Latency test lines were observed during the survey and the resultant latency was determined to be 0 seconds due to time synchronization between the GPS receiver and PC clock.

E.4.2. BAR CHECK

To confirm the draft of the sonar transducer, a bar check observation was recorded by lowering a metal plate to a known depth from the water surface. The observed bar check depth was compared to the logged single beam depths. The logged single beam depths showed agreement within 0.00 feet during the observation. In addition to the bar check, a minimum accurate depth of 1.3 feet was also determined, by using a sounding pole value compared to the draft corrected raw sonar depth.

E.4.3. SOUND VELOCITY

Detailed measurements of the sound velocity profile through the water column are crucial in producing accurate hydrographic surveys. The sound velocity profile of the water column was measured using Odom Digibar Pro several times each day. The average sound velocity for each profile was entered into the echosounder. The average sound velocity of the water column with river mile, cast position and date are listed in Table 2.

Table E. 2. Sound Velocity Measurements

Cast	River Mile	Northing	Easting	Average Sound Velocity (feet per second)	DATE
SVP-2	1.30	291915	1034138	4821	8/10/2009
SVP-3	0.00	286528	1027327	4859	8/10/2009
SVP-4	0.50	286716	1028455	4878	8/10/2009
SVP-1	0.20	286600	1029760	4844	8/11/2009
SVP-2	0.40	288063	1030418	4799	8/11/2009
SVP-3	0.60	289102	1032698	4802	8/11/2009
SVP-4	1.70	293227	1033421	4808	8/11/2009
SVP-5	2.25	294607	1030778	4810	8/11/2009
SVP-6	2.85	295769	1027624	4806	8/11/2009
SVP-1	3.00	296229	1027260	4796	8/12/2009
SVP-2	1.10	290975	1034135	4813	8/12/2009
SVP-1	3.50	299033	1026401	4797	8/13/2009
SVP-2	4.70	304483	1028647	4807	8/13/2009
SVP-3	5.75	309905	1028423	4812	8/13/2009
SVP-4	6.80	315642	1028940	4808	8/13/2009
SVP-1	7.40	317117	1031569	4791	8/14/2009
SVP-2	8.30	321010	1033224	4793	8/14/2009
SVP-3	9.30	324442	1029750	4792	8/14/2009
SVP-1	10.05	326879	1030445	4816	8/17/2009
SVP-2	11.20	331819	1030780	4826	8/17/2009
SVP-3	12.50	336139	1034133	4827	8/17/2009
SVP-1	12.80	336570	1032947	4813	8/18/2009
SVP-2	13.80	338243	1034322	4821	8/18/2009

SVP-3	14.90	344096	1034756	4831	8/18/2009
SVP-4	16.10	349911	1031183	4827	8/18/2009
SVP-1	16.30	349662	1030170	4824	8/19/2009
SVP-2	17.50	354192	1031282	4827	8/19/2009
SVP-3	18.80	360491	1030859	4831	8/19/2009
SVP-4	19.70	364585	1029011	4817	8/19/2009
SVP-5	20.20	367339	1029714	4810	8/19/2009

E.5. DATA PROCESSING

Post-processing of the single beam data was conducted utilizing Hypack single beam editor. The latency test was analyzed from several latency tests and no latency correction was necessary. Water surface elevation data was applied to adjust all depth measurements to NAVD-88 elevations. The average sound velocity of the water column, entered in to the Odom Echotrac CV100 echosounder during data acquisition, was verified from the records of the Odom Digibar Pro. Sounding and position data was reviewed and edited for data flyers.

After the data was reviewed and edited, it was decimated to a density of approximately 2 feet along the cross-section and exported from Hypack as an ASCII point file, containing the Easting, Northing and corrected Elevation. The bathymetric ASCII point file, topographic data and planned cross-section alignment were imported into Terramodel software for final review and compilation of final deliverables. In many cases additional or redundant data was collected along the cross-section to verify data quality, although overlapping data was eliminated in the final composite cross-section.

E.5.1. DATA EXPORT

The final data set was exported from Terramodel using a station and offset report function relative to each cross-section designed alignment. The station and offset report was generated as an ASCII text file and was imported into Microsoft Excel to parse out the unwanted data fields generated by the Terramodel station and offset report and to generate the final deliverable ASCII files that met project specified format requirements.

E.5.2. DELIVERABLES

Deliverables consist of the following:

- Individual cross-section data provided as a comma delimited text file formatted to adhere to USACE standard coordinate file coding system described in Chapter 12 of Engineering Manual 1110-1-1005 (01 Jan 07).
- • Digital photos (six from each cross section) with file names using the cross-section name and the feature shown in the photo.
    - o 520LA is cross-section 520 from the left bank across towards the right bank
    - o 520LD is cross-section 520 from the left bank looking downstream
    - o 520LU is cross-section 520 from the left bank looking upstream
    - o 520RA is cross-section 520 from the right bank looking towards the left bank

- o 520RD is cross-section 520 from the right bank looking downstream
- o 520RU is cross-section 520 from the right bank looking upstream

Appendix F. Review Comments

F.1. LEVEE REVIEW

Comment Report: All Comments
Project: Cowlitz Levees SWL
Review: Final Study ATR Report
Displaying 13 comments for the criteria specified in this report.
422 ms to run this page

Id	Discipline	DocType	Spec	Sheet	Detail
2919346	Geotechnical	Technical Report	n/a'	n/a	n/a
pg 3, para 2 - are all slip surfaces assumed to be circular? Submitted By: John Gent (509 527 7610). Submitted On: 08-Dec-09					
1-0	Evaluation **For Information Only** Yes, only circular slip surfaces were used. Submitted By: Jeremy Britton ((503) 808-4851) Submitted On: 11-Dec-09				
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: John Gent (509 527 7610) Submitted On: 05-Jan-10				
	Current Comment Status: **Comment Closed**				
2919348	Geotechnical	Technical Report	n/a'	n/a	n/a
pg 3, para 4 - is the "likely" failure a result of water inflow or breaching? Submitted By: John Gent (509 527 7610). Submitted On: 08-Dec-09					
1-0	Evaluation **For Information Only** Good point. The intent was that breach failure is likely. Upon further consideration, a very short overtopping duration may not cause breach, but may cause undesirable water inflow. Will revise to define "failure" as either water inflow or breaching. Submitted By: Jeremy Britton ((503) 808-4851) Submitted On: 11-Dec-09				
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: John Gent (509 527 7610) Submitted On: 05-Jan-10				
	Current Comment Status: **Comment Closed**				
2919360	Geotechnical	Technical Report	n/a'	n/a	n/a
General Comment - applies to sections 2 thru 6 limited subsurface information appears to vary from about 300 ft between explorations (Castle Rock) to over 1200 ft between explorations. A different adjective is appropriate Submitted By: John Gent (509 527 7610). Submitted On: 08-Dec-09					
1-0	Evaluation **Check and Resolve** Are you referring to adjective "limited"? Thanks. Submitted By: Jeremy Britton ((503) 808-4851) Submitted On: 11-Dec-09				
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: John Gent (509 527 7610) Submitted On: 05-Jan-10				
	Current Comment Status: **Comment Closed**				

2919367	Geotechnical	Technical Report	n/a'	n/a	n/a
General Comment much of the levees are characterized by a few cross-sections and analysis. while not trying to raise the specter of "characteristic length," are the cross-sections truly significant for complete stretches of levee? Submitted By: John Gent (509 527 7610). Submitted On: 08-Dec-09					
1-0	Evaluation **Concurred** The levee in each reach has similar characteristics. The concept of characteristic length was not used. I believe the characteristic length approach would be too sophisticated for the simplified pseudo-reliability approach used for the study. Submitted By: Jeremy Britton ((503) 808-4851) Submitted On: 11-Dec-09				
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: John Gent (509 527 7610) Submitted On: 05-Jan-10				
	Current Comment Status: **Comment Closed**				
2919377	Geotechnical	Technical Report	n/a'	n/a	n/a
General Comment When the probability of failure is zero, the factor of safety against global instability is 1.4, the exit gradient is less than 0.5, and the riverside is protected against scour. it seems there is untapped capacity. Submitted By: John Gent (509 527 7610). Submitted On: 08-Dec-09					
1-0	Evaluation **Concurred** The "untapped capacity" is needed to account for uncertainty in the soil properties and other conditions affecting the failure mechanisms. This is part of simplified pseudo-reliability approach: that meeting the required FS (e.g. 1.4 for slope stability) is necessary to ensure zero probability of failure, given uncertainties involved. Submitted By: Jeremy Britton ((503) 808-4851) Submitted On: 11-Dec-09				
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: John Gent (509 527 7610) Submitted On: 05-Jan-10				
	Current Comment Status: **Comment Closed**				
2919380	Geotechnical	Technical Report	n/a'	n/a	n/a
pg 37, para 2 - is the block wall "rigid" or is it just stacked? if it is just stacked, was overturning and sliding checked? many block walls will fail w/active earth pressures. As hydrostatic pressures will be larger, it is likely that water may have a significant affect. Submitted By: John Gent (509 527 7610). Submitted On: 08-Dec-09					
1-0	Evaluation **Concurred** Wall is stacked blocks. Concur about impact of water pressure on wall. The grade at which probability of failure = 0 is less than 6 inches onto wall. In other words, it is assumed wall can tolerate up to 6 inches of water load without failing. Because of the failure mechanisms you mentioned, the wall is not counted on with more than 6 inches of water on it. Submitted By: Jeremy Britton ((503) 808-4851) Submitted On: 11-Dec-09				
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: John Gent (509 527 7610) Submitted On: 05-Jan-10				

	Current Comment Status: **Comment Closed**				
2919381	Geotechnical	Technical Report	n/a'	n/a	n/a
pg 37, para last - is the "factor of safety against sliding" actually the factor of safety against global instability? Submitted By: John Gent (509 527 7610). Submitted On: 08-Dec-09					
1-0	Evaluation **Concurred** Yes. Factor of safety against global slope instability. Submitted By: Jeremy Britton ((503) 808-4851) Submitted On: 11-Dec-09				
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: John Gent (509 527 7610) Submitted On: 05-Jan-10				
	Current Comment Status: **Comment Closed**				
2919383	Geotechnical	Technical Report	n/a'	n/a	n/a
pg 40, section 5.2, para 2 - will heavy equipment be needed to handle the concrete structures? if so, are they prepositioned and/or dediciated? Submitted By: John Gent (509 527 7610). Submitted On: 08-Dec-09					
1-0	Evaluation **Concurred** Yes, heavy equipment will be needed. The concrete structures are positioned near the tunnel entrance, but the heavy equipment needed to handle them are not prepositioned. The County has equipment that can be mobilized to the location to handle the structures. Submitted By: Jeremy Britton ((503) 808-4851) Submitted On: 11-Dec-09				
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: John Gent (509 527 7610) Submitted On: 05-Jan-10				
	Current Comment Status: **Comment Closed**				
2919388	Geotechnical	Technical Report	n/a'	n/a	n/a
pg 41, section 5.2.1, para 2 - this is the only section you put a time on the inudation. Further, is this the authorized level? Submitted By: John Gent (509 527 7610). Submitted On: 08-Dec-09					
1-0	Evaluation **For Information Only** I'll include authorized levels of protection in report. They are 143 years for Kelso, 167 years for Longview, 167 years for Lexington, and 118 years for Castle Rock. Submitted By: Jeremy Britton ((503) 808-4851) Submitted On: 11-Dec-09				
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: John Gent (509 527 7610) Submitted On: 05-Jan-10				
	Current Comment Status: **Comment Closed**				
2919390	Geotechnical	Technical Report	n/a'	n/a	n/a
pg 54, para 1 - railroad embankment is seldom made of sand.					

Submitted By: John Gent (509 527 7610). Submitted On: 08-Dec-09

1-0	Evaluation **Concurred** Okay. This is something we'll have to investigate further. Sand was observed near the surface but it is surely possible that that was not representative of the bulk of the embankment. This will not change judgment about expected performance of very broad embankment. Submitted By: Jeremy Britton ((503) 808-4851) Submitted On: 11-Dec-09
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: John Gent (509 527 7610) Submitted On: 05-Jan-10
	Current Comment Status: **Comment Closed**

2919391	Geotechnical	Technical Report	n/a'	n/a	n/a

pg 54, section 5.3, para 2 - are the necessary resources available to raise low spots given an event?

Submitted By: John Gent (509 527 7610). Submitted On: 08-Dec-09

1-0	Evaluation **Concurred** Yes. The county has plan. Low spots were successfully raised during 1996 event. Submitted By: Jeremy Britton ((503) 808-4851) Submitted On: 11-Dec-09
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: John Gent (509 527 7610) Submitted On: 05-Jan-10
	Current Comment Status: **Comment Closed**

2919407	Geotechnical	Technical Report	n/a'	n/a	n/a

pg 53, para 1 - the description indicates dredged material was placed next to levee. no mention is made of the material's scour resistance.

Submitted By: John Gent (509 527 7610). Submitted On: 08-Dec-09

1-0	Evaluation **Concurred** There is riprap on the riverward slope beneath the dredge sand. See Drawing CZR-7-5/6 in Appendix B. There is also substantial dredge sand placed landward of the levee, making the "levee" quite broad. Submitted By: Jeremy Britton ((503) 808-4851) Submitted On: 11-Dec-09
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: John Gent (509 527 7610) Submitted On: 05-Jan-10
	Current Comment Status: **Comment Closed**

2919412	Geotechnical	Technical Report	n/a'	n/a	n/a

on the seepage analysis plots, the equipotential lines extend above the phreatic surface. is this just an artifact of the software?

Submitted By: John Gent (509 527 7610). Submitted On: 08-Dec-09

1-0	Evaluation **For Information Only** Seep/W models flow above the phreatic surface. User has to make assumption about permeability of

	soil in unsaturated region. So those equipotential lines above phreatic surface are "real" to Seep/W; the flow is insignificant though because I made the permeability very small in the unsaturated region. Submitted By: Jeremy Britton ((503) 808-4851) Submitted On: 11-Dec-09
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: John Gent (509 527 7610) Submitted On: 05-Jan-10
	Current Comment Status: **Comment Closed**

F.2. HYDROLOGY REVIEW

Comments on Cowlitz flow frequency analysis
Hydrologic Engineering Center, November 2009
Beth Faber

I found the frequency analysis to be performed very well. It followed guidance where it was available and adequate, and developed creative solutions to problems that were not well addressed in guidance. Overall, the resulting regulated frequency analysis is strong.

This study, because it was for a small but high profile basin, had the opportunity to experiment with new techniques for difficult issues such as the creation of synthetic flood events. The study added to existing methods by developing a new approach to visualizing and describing historical basin-wide events, and summarizing that information to create an array of synthetic storm patterns that vary spatially and temporally. The approach is well-thought out and strong and should be applicable to other basin-wide studies, though perhaps not to the level of detail possible in this small basin.

The document is well written, and describes the steps and approaches in the analysis much clearly than other documents of its kind.

These comments are based on an early draft of the document, and the reviewer has since received a later version of the LOP report. Therefore, in many places, the "status" column for the individual comments shows a comment has already been addressed. There are probably more already addressed that I did not mark, and the authors will make note of those.

Some of the comments are questions, and might or might not lead to changes in the document. Some of the comments on the first version might refer to text that has subsequently been removed.

Comments on first version

	Section	Page	Paragraph	Sentence	Comment	Status
1					Overall, the figures and tables are not referenced by name in the text. In the final version, each should be mentioned my name.	Added figure and table numbers to text.
2	January 2009 Event	2	1		Are the 2 1996 events independent? (Did the reservoirs stay in the flood pool between events?) I assume they were thought to be independent since both were maintained in the data set.	Yes. The two storms were separated by approximately 2 months. There was sufficient time to draw the reservoir back down to the rule curve prior to the second storm.
3		2	3		The regulated flow for 2009 was based on having a great deal of empty storage space available (twice the required). Using the value as-is is appropriate for a regulated flow frequency analysis. However, when doing an unregulated flow frequency followed by reservoir modeling, the event would likely have been simulated with less flood space available. The regulated frequency method used here does incorporate the uncertainty in starting storage more completely, but as this is one of the largest flows of the record, the fact that this lowers the upper end of the frequency curve should be kept in mind.	Partially agree – the 2009 event was so strongly dominated by runoff from below Mossyrock that the same peak regulated discharge would have occurred at Castle Rock even without the additional flood storage available. This storm is an excellent example of the impact of spatial variability.
4	Pre-1969 gage data	2	Figure 2		the colors are backwards in the legend	FIXED
5		3	2		final sentence: suggest adding to the end "…more able to represent the basin into the future."	Unclear which paragraph is being described… pending as of 14 DEC 2009
6			Figure 3		Some events seem to show higher regulated than unregulated flow. Why?	Explanation: regulated discharges are occasionally higher than the unregulated discharges in (Figure B.9) because of several reasons: 1) because releases from Mossyrock occasionally exceed inflows to the reservoir during small to medium sized events, 2) unregulated discharges are daily average and the regulated discharges are instantaneous, and 3) occasional lack of short interval data results in

						course unregulated discharge modeling results.
					Suggest labeling horizontal axis as "unregulated discharge" rather than "natural discharge"	Done
7			Figure 4		Would help to include a legend, and label the largest events with their year. It would also help to include the unregulated frequency curve as well.	Done
8	1934 Event	6	2	last	The phase "assumes full storage capacity at the beginning of the event" is confusing. I guess this means all capacity is empty and available, but sounds like the space is full.	Clarified
9					Mention that the design only looks at the 2nd wave of the 1934 event. It is clear whether this was adequate, or if it impacted the result?	Done
10		6	4	2nd	recommend changing "daily and annual peak" to "daily and instantaneous peak"	Done
11		7	2		recommend showing ResSim output graphic – storage, inflow, outflow	DONE
12	Discharg e Frequenc y Curves	8	2	2nd	Says low outliers were excluded to create a better graphical fit of the upper end of the curves. I would remove the word "graphical" and just say "better fit."	Done
13		8	5	last	Does this refer to a regression relationship for literally generating missing flows, or a 2-station comparison to adjust sample statistics?	Unsure of technical description. Currently seeking guidance… pending as of 14 DEC 2009
14		9	6	1st	Suggest "Frequency analyses were then performed to create Log Pearson III VDF curves…"	Done
15				2nd	Suggest change "estimated" to adjusted"	Done
16		10	1	1st	Hourly hydrographs were "reduced to duration maxima and translated to AEPs using the VDF curves." Which durations were used to choose the AEP? Was a most relevant duration chosen?	All available duration-AEPs (1- through 7-day) were used to verify the frequency statistics – no reduction or assumption of relevance was made.
17			3		Suggest spelling out AAP, average annual precip, once.	Done
18					Mention that the regression relationship produced strong correlations. In regression, we're more interested in which generates the smallest standard error, not the highest correlation or R^2. Did the chosen relationship have the smallest standard error?	"Strong correlations" is used in the next. And yes, smallest standard error was implied but not stated.

19					Suggest "mean of log discharges" rather than "log mean of discharges"	Done
20		11	Figure 6		The axes are labeled as either side of the equation on the previous page. This implies equality and so the 45-degree line. Is something mislabeled? If not, please explain in the text. Also, suggest reporting the x values	Three variables are varied on the x-axis
21	Storm Patterns	11	1	2nd	Suggest changing "converted to AEPs" to "AEPs were estimated"	Done
22					"Weibull" is misspelled. (Note, check other locations in the document)	Noted
23		12	Figure 7		Excellent graphic! And a very good way of summarizing the information and viewing the events.	Thanks
24		12	1		Suggest choosing a duration of interest rather than averaging the AEP across several durations.	Average AEP was used for patterns analysis, but single duration-AEP's were later used to specifically define a frequency of an event. See discussion in section B.4.5.
25		14	2	Last	Say that the array of storms is meant to only span the range of flow for a given probability. If choose to assign equal probabilities (or unequal probabilities) to the storms, could combine them to a best estimate using the total probability theorem.	DONE
26	Synthetic Hydrographs	16	1		Misspelled author name "Hickey."	Done
27	Reservoir Routing	17	Figure 17		There's not much info in a ResSim schematic. Maybe add in a view of a reservoir editor or a routing reach…	Unsure if more ResSim description is required… question pending as of 14 DEC 2009
28	Synthetic Hydrograph Results	20	Figure 15		Suggest adding confidence intervals. Could use the +/- 2 standard deviation bounds suggested from FDA once the curve is entered.	Done

Additional comments on the later version

	Section	Page	Paragraph	Sentence	Comment	Status
29	Overview	2 - 3	First figure and table		The figure and the table don't seem to quite match.	YES! We had the incorrect values in the final table. Discharges corrected in Table 5 (section 3.5), Table

						B.6.(section B.5) and B.9 (Section B.6.).
30	December 1933 event	12	Figure B7		Mentions that the oscillations are just a minor glitch in the model that have no impact on downstream flow. Has this been verified? Can the glitch be addressed?	Yes, it has been verified that this instability does not affect the results at Castle Rock. It should be noted that the oscillation was only occasionally present, depending on the regulating scheme. A solution that would work for all regulating schemes could potentially be created with more advanced ResSim rule programming; however, since it does not affect the results, it is not considered to be a worthwhile exercise.
31	All other pre-regulation water years	14	Figure B10		The symbol for m=0.85 is missing from the legend, unless "estimated" refers to m=0.85. If so, please make this clearer.	Done
32	Historical Data	15			How was the historical period applied? Adjusted plotting positions?	Added explanation of the application of the historical period.
33	Synthetic Hydrology Results	28	2		From the description, sounds like the combination of the synthetic curves with the total probability theorem was done right. But should describe it by name, or include the equation. Also, with interpolating probability, better to a probability scale rather than a log scale (meaning, interpolate with the standard normal deviate for a given probability.)	Added clarification.

F.3. HYDRAULIC REVIEW

Hydraulic Review was performed by Omaha District (NWO) personnel the week of 30Nov2009. Comments were received in three documents: A Omaha standard HEC-RAS modeling checklist "HYD 6", a memo entitled "Review of Hydraulic Certainty" and editorial comments provided via pdf. Comments made in these three documents are combined in this document along with Portland District responses.

Hydrologic Engineering Branch Checklist HYD 6

HEC-RAS Modeling

The Cowlitz River HEC-RAS model was reviewed by Curtis Miller on 1-3December 2009.

Notes in green indicate no action needed.
Notes in gold indicate consideration of revisions or double checking data is recommended but changes are not necessarily required.
Notes in red indicate items that should be addressed or review items that could not be verified.

Overall Model Review Requirements.

1. A map (at least a work map) showing the cross section location, cross section numbering, flood boundaries should be prepared and submitted for the peer review. This will provide the peer reviewer with a better understanding of the overall model and its layout. The map should be maintained with the project file or calculations file.

NWO Comment: OK

2. Bridge plans and/or photos should be submitted with the work map for the peer review. This data should also be maintained with the project file or calculations file.

NWO Comment: No bridge plans available for review.
NWP Response: With the exception of the new bridge at Lexington, constructed in 2007-2008, bridge data has been brought forward from the older models and verified by additional photography, LiDAR, aerial photography and field investigation. In the case of the Lexington bridge, a field investigation provided the information unavailable from LiDAR.

3. A draft hydraulics analysis should be prepared and submitted for the peer review. This will provide the peer reviewer with documentation as to how the model was prepared and what assumptions went into assigning various hydraulic parameters.

NWO Comment: OK

Model Construction.

1. Channel stationing – Check reach length compared to stations and overall project length.

NWO Comment: OK

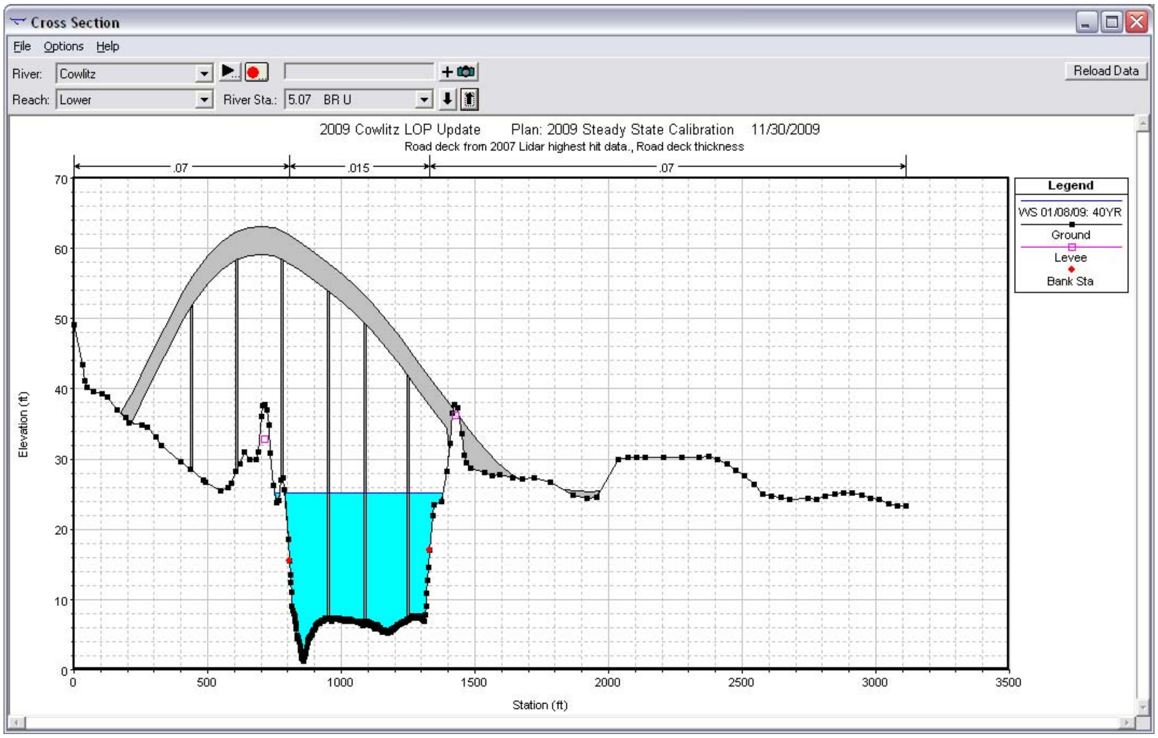
2. Roughness – Check if using channel roughness, horizontal, vertical. Verify roughness values are reasonable and for continuity through the project. Verify roughness varies with different project areas (urban, lined channel, vegetated channel) and material types (sand, silt, cobble, riprap). Consider flow depth impact on roughness.

NWO Comment: Uses LOB, CH, ROB roughness.

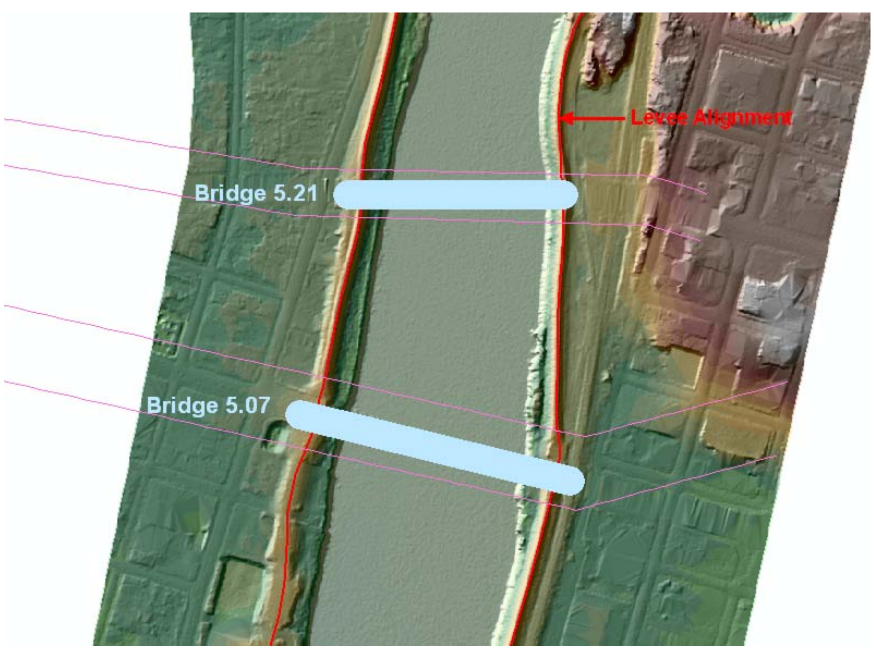
The roughness values in the downstream reach (n=0.015 from RM 0.0 to 6.75) appear low, even considering the transition to upper regime/planar bed. The difference between the upper reach and lower reach is not clear (i.e. since the discharges do not differ substantially over the entire reach, why is the lower reach in transition while the upper reach uses more typical roughness values? Is it because of the change in stream slope? This should be explained in the report. Section C.5.4, page 131, notes that van Rijn's method for predicting bed regime was applied to the lower 8 miles of the river. Was this also done for the upper 12 miles? If so, what were the results?

Using only one event for calibration does not ensure model accuracy for the entire range. If the value of 0.015 is in fact correct for the calibration event, Figure 4-1 would suggest varying roughness values over the range of discharges. Consideration should be given to using flow roughness factors within HEC-RAS to account for the changing roughness. Consider calibration to gage data over the entire range of available discharges using flow roughness factors. If calibration to gage data has been evaluated, results should be presented in the report.
NWP Response: Please see response to Model Calibration\Verification comment 1.
NWO Backcheck: **Based on data presented in the response to Model Calibration/Verification, comment 1, COMMENT CLOSED.**

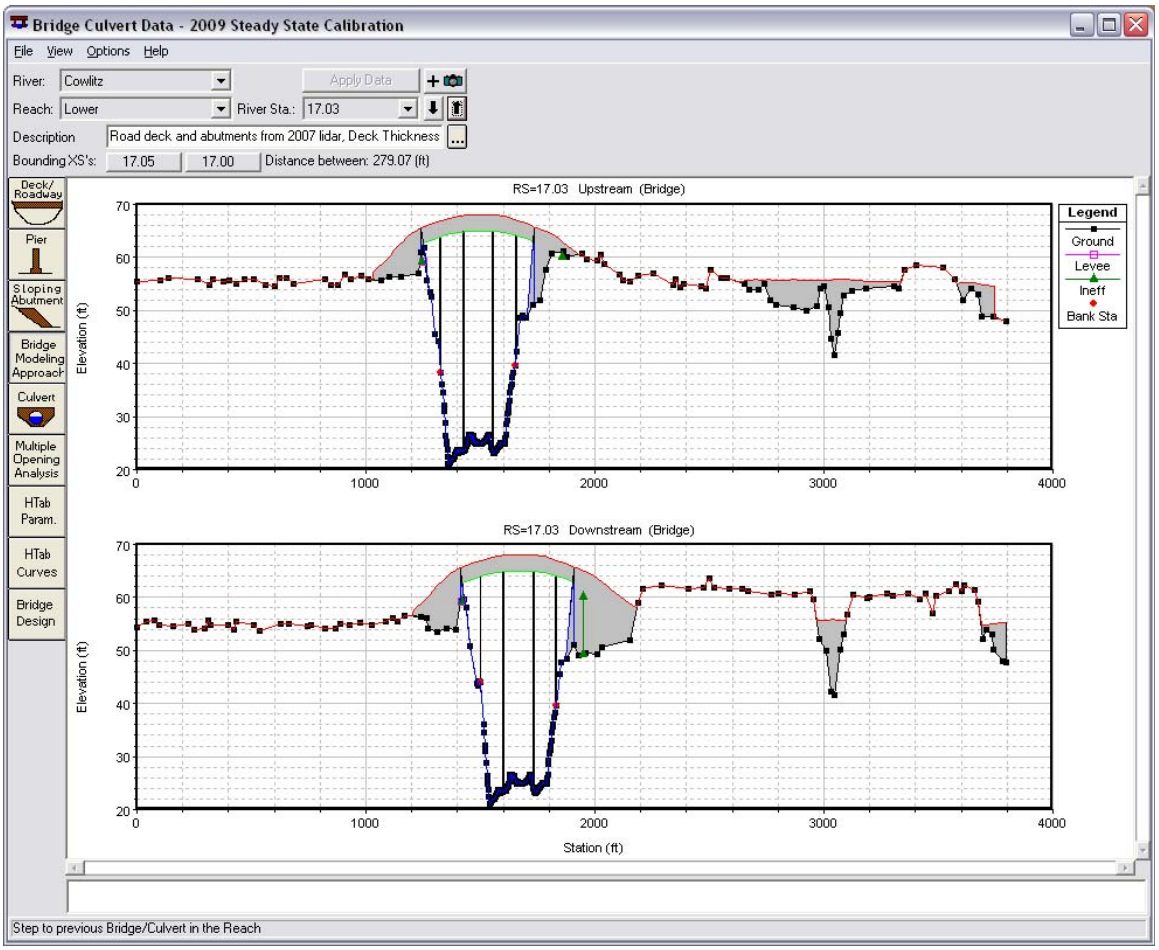
3. Expansion/Contraction – Check variation near structures.
NWO Comment: Expansion/Contraction coefficients were not varied around bridges. Appendix C, section C.3 notes that the downstream 20 miles of the Cowlitz are highly leveed and do not require adjustment to default C/E coefficients. However, a sensitivity run with the C/E coefficients increased from 0.1/0.3 to 0.3/0.5 around the bridges shows impacts to the computed water surface elevation (average of 0.28 ft increase over the entire reach with a maximum increase of over 1ft). C/E coefficients are not necessarily a calibration parameter—their impact to computed water surface elevations should be evaluated based on geometry of the stream. The impact to the computed water surfaces from varying the C/E coefficients indicates expansion and contraction are occurring, especially around the bridges.
NWP Response: NWD stands by its decision to keep expansion/contraction coefficients at default values. The "highly leveed" comment in the report may mislead the actual intent of the statement. In the lower 10 miles of the river, which contains 6 of the 7 bridges, levees are located immediate to the river on both the left and right banks. Bridge abutments are minimal to non-existent within the leveed cross section. The bridge at RM 5.07 pictured below is a typical bridge example on the lower 10 miles. Also below is LiDAR mapped with levee location and bridge location. There is no additional contraction or expansion of the flow due to the bridge or abutments at these locations short of the piers. The additional losses due to bridge piers are included by using the highest energy answer of Yarnell and Energy (standard step) to determine losses.

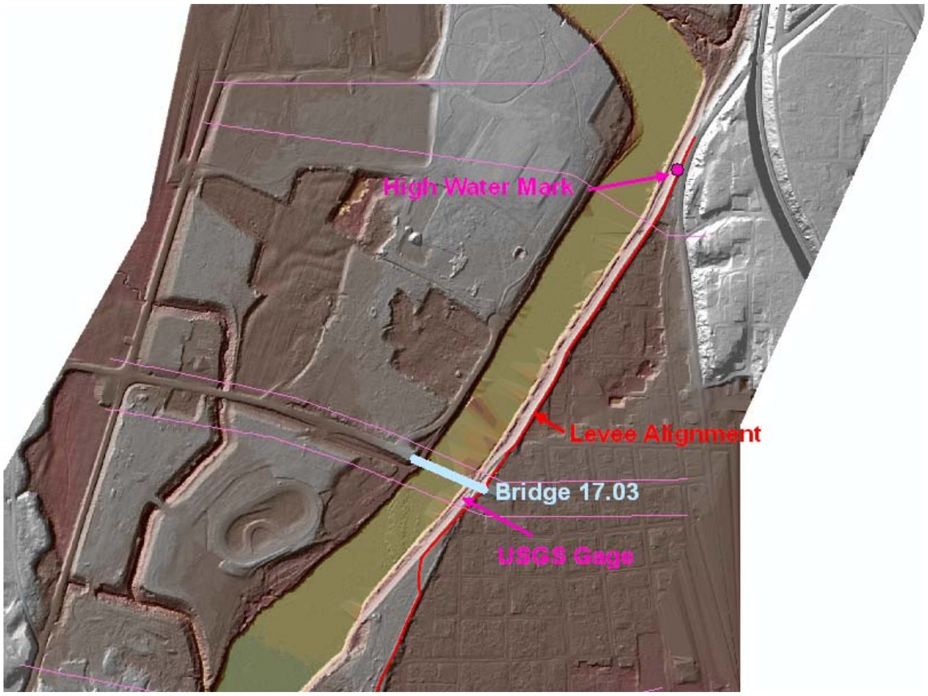


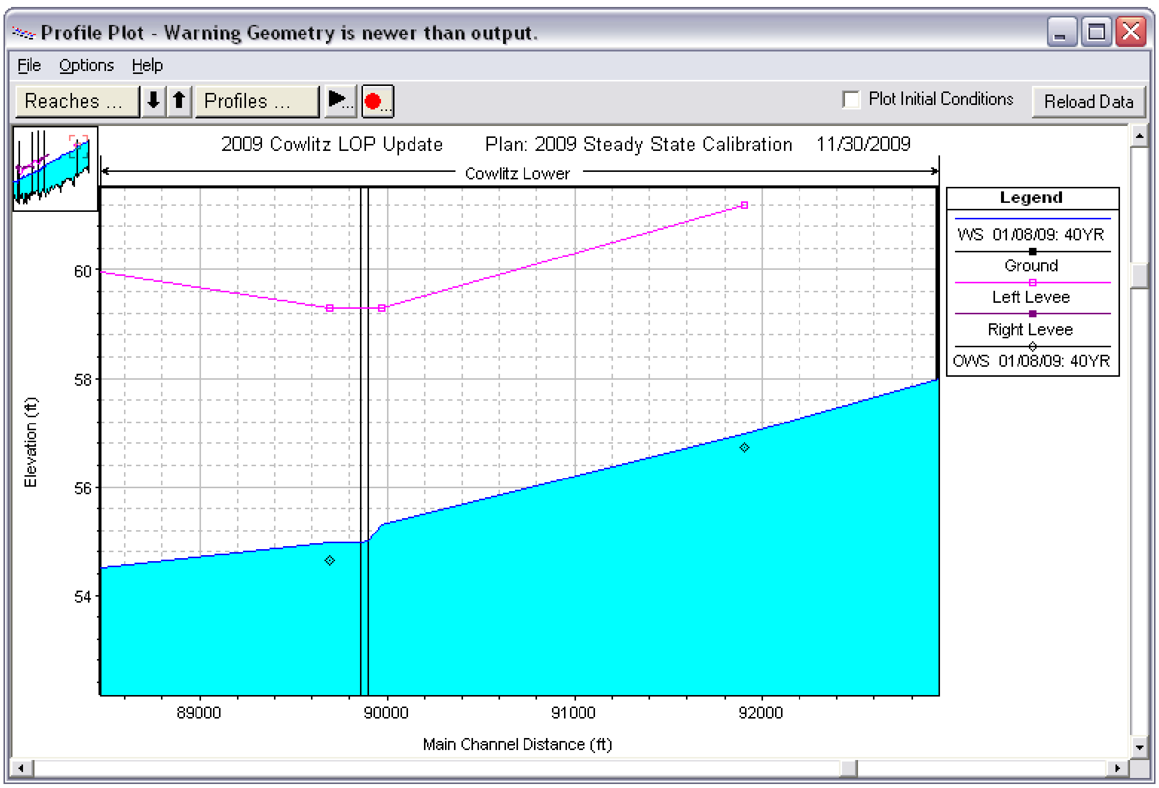
The alignment below, typical on the lower Cowlitz River differs significantly from the research bridges used to develop the recommended expansion and contraction coefficients. The research bridges constricted the overall floodplain width between 50 and 90% according to Appendix B of the *HEC-RAS River Analysis System Hydraulic Reference Manual*. For these reasons we feel that 0.1 and 0.3 are appropriate contraction and expansion loss coefficients near bridges in the lower 10 miles on the Cowlitz River.



The remaining upstream bridge at RM 17.03 is the only bridge where a typical contraction of the floodplain may exist to warrant increases in the default expansion and contraction coefficients. This is confounded with complex flow patterns around high dredge spoil piles in the overbank such that bridge contraction is no different than that of the surrounding topography. Appendix B of the *HEC-RAS River Analysis System Hydraulic Reference Manual* states in the *Recommendations From the Study* section that, "the evaluation of contraction and expansion coefficients should ideally be substantiated by site-specific calibration data, such as stage discharge measurements just upstream of the bridge." High water marks upstream and downstream side of the bridge were collected following the calibration event, also shown below. The calibrated profile is approximately 0.2ft above both high water marks indicating that the model is accurately reproducing the energy loss at the bridge for an event that was out of the river banks.







NWO Backcheck: **COMMENT CLOSED**

4. Verify location of bank stations. Bank stations should correspond with roughness values.
NWO Comment: OK

5. Verify location of encroachments and ineffective flow areas. Check location with project geometry. Evaluate for range of flow events and consider flow depth. Generally encroachments should not be within bank stations.

NWO Comment: Verify effective flow for larger events in right overbank, RS 11.65-12.40. Right side of cross section does not tie off for 10-100yr events depending on cross section.
Other than the above mentioned area, ineffective flow areas appear reasonable, especially considering the extreme range of events being modeled with one geometry.

NWP Response: The right side of cross sections 11.65 through 12.40 do not tie off due to a valley related to a tributary on the right bank as shown below. The area to the west of the roadway is assumed to be largely ineffective due to its recession into the valley wall and the presence of the roadway.

NWO Backcheck: **Concur. COMMENT CLOSED**

6. For multiple reaches check model connectivity.

NWO Comment: N/A

7. Check the cross section layout to ensure the cross sections were taken perpendicular to the flow path.

NWO Comment: OK

8. Levees

NWO Comment: Verify levee elevations at index points. Is there a reason why the levee elevations are not consistently set to the safe water level? (A sensitivity was conducted to determine impact of setting

levees to safe water level at index points—no impact to computed water surfaces until the 0.1% chance exceedance event, and even then, it was minor.)
NWP Response: Levee elevations used in the model are all "top of levee" elevations. The elevation data was taken from a recent levee inventory effort that consisted of point data spaced approximately every 100 ft along the top of each levee. All data points were assigned to their closest cross section and sorted by elevation. The lowest elevation was assigned as top of levee for the particular cross section. The purpose of using top of levee in lieu of safe water elevation for hydraulic modeling overtopping criteria is related to creating rating curves that reflect a system without and risk for seepage or other through-levee failure mechanism and therefore a more conservative rating curve to the top of levee elevation. The effects of this assumption for the system are minimal as shown in your sensitivity analysis.
NWO Backcheck: **Concur, COMMENT CLOSED.**

Model Flow.

1. Check design flow from Hydrology.

NWO Comment: OK. It is unclear why the equivalent years of record (90 years) is greater than the 83 years of data (actually 42 years of measured data, 42 years of estimated data). Table 4-5 of EM 1110-2-1619 indicates using a maximum of the systematic record length for an analytical distribution fitted with long-period gauged record, or a percentage thereof.
NWP Response: The additional equivalent years of record are due to knowledge of historic flood event prior to the beginning of annual record keeping. Investigation of historical floods on the lower Cowlitz indicates that the largest flood observed in the systematic record period (observed regulated event of 1996) is the largest event dating back to 1896. The additional 30 years of historical record increases certainty but at a significantly reduced value, estimated at 50% for this analysis. An additional 15 years of EYR are added to the 75 years representing the observed period of record for a total of 90 years representing the EYR of this hydrologic study. This method had been vetted thorough the hydrologic reviewer, Beth Faber at HEC.
NWO Backcheck: **Explanation is valid. While review of the hydrology was outside the scope of this review, section 3.4 of the main report is somewhat confusing: "…(EYR) of 90 years is calculated based on 83 years of data adjusted for the historical period and for additional uncertainty…" After reading through section B.7, the methodology is understood. Perhaps a sentence or two could be added to section 3.4 for futher explanation? This question was more out of curiosity than anything; no response to this comment is necessary. COMMENT CLOSED**

2. Check flow change locations.

NWO Comment: OK

3. Check for split flow areas as necessary.

NWO Comment: N/A

Model Boundary.

1. Check boundary conditions. Verify slope, assumed water surface, etc.

NWO Comment: OK-known water surfaces from Columbia River stage-frequency assuming coincident peaks

2. Check boundary condition impact on computed results with a sensitivity analysis.

NWO Comment: Using coincident peaks on Columbia River for d/s boundary condition results in conservative estimate of LOP in lower reach. A sensitivity analysis using a starting WS 1ft higher than estimated was conducted. All profiles converged to calibrated model at or before RM 8.0.
NWP Response: Coincident peaks is a conservative assumption that has historically been made in the lower Cowlitz level of protection analyses. No level of protection issues arise using the conservative

assumption in the area that would be affected. For purposes of economizing the analysis, a coincident analysis was not pursued in 2009.

Bridge\Culvert Data Input.

1. Using bridge plans or photos check if the bridge model has the appropriate number of bridge openings, number of piers, etc.

NWO Comment: No bridge plans available for review.

2. Check all input data including pier location, top of road, bridge low chord, invert, and abutments.

NWO Comment: No bridge plans available for review.

3. Verify roughness through bridge.

NWO Comment: OK

4. Check location of cross sections and encroachments for effective flow.

NWO Comment: OK

5. Check all modeling parameters including low flow method, high flow methods, and all coefficients (momentum, pier shape).

NWO Comment: OK. Could not verify pier shape coefficients.
NWP Response: Pier shapes have been verified by NWP personnel.

Bridge\Culvert Model Results.

1. Check bridge summary table for flow type, weir flow, pressure flow. Verify that it corresponds with data input assumptions.

NWO Comment: OK

2. Verify that weir and pressure flow are modeled correctly for the bridge geometry and results. Verify that weir flow can actually occur (energy grade vs. water surface).

NWO Comment: OK

3. Check for profile crossing upstream of structure for range of events.

NWO Comment: Profile crossings occur d/s from BR 1.35. However, the crossings are for extreme events (1000yr+). OK
NWP Response: Utilizing a single geometry for the large range of events resulted in the anomaly noted; crossing profiles at BR1.35. As noted this is seen at a large event that is far beyond the 100% certainty of levee failure and does not affect the level of protection.
NWO Backcheck: **Concur. COMMENT CLOSED**

4. For pressure flow, perform hand calculation check based on open area. Check with nomographs or other culvert data. Verify that pressure flow increases with increasing head.

NWO Comment: N/A

5. Check overbank flow vs. weir flow vs. channel flow through the bridge sections.

NWO Comment: OK

6. Check for reasonable velocity through the bridge.

NWO Comment: OK

7. Check for reasonable head loss through the bridge sections.

NWO Comment: OK

Model Calibration\Verification.

1. Check calibration data for accuracy. A profile plot may illustrate bad data.

NWO Comment: Data used for calibration (HWMs and gage data from Jan2009) event appear reasonable.

It is highly recommended that model results be compared against a range of discharges rather than a single event. Plotting computed results against the available gage data could illustrate the validity of the roughness values used to calibrate the model to the Jan2009 event. While the model results compare favorably against the one event, comparison to gage data may show separation from observed rating curves over the entire range of discharges. While most of the observed data is likely less than the authorized LOP discharges, the uncertainty evaluation takes the entire range into account. If this analysis has already been conducted, it should be clearly presented in the report.

It is agreed that the roughness during the relatively high flow event could be low, but it is not clear that Figure C.5, based on a river in Bangladesh, provides adequate justification for using a roughness value that approaches that of concrete. The conclusion in section C.5.4, page 131, that "a review of the hydraulics for the lower 8 miles of the river using Van Rijn's method for predicting bed regime provides evidence of regime changes around the calibration event" should contain additional support. From Figure 4-3, page 19, it appears that the regime change occurs at a much higher discharge than that of the calibration event. Is this interpretation correct?

The low roughness value (0.015) necessary to nearly match the observed data from the Jan2009 event brings up several questions that should at least be addressed in the writeup (note that to exactly match the HWMs, an even lower roughness value is required):

- Are the discharges in the lower reach accurate and fairly certain? The roughness values used in the upper reach indicate a reasonable estimate of the discharge from the Castle Rock gage. However, is it possible that flow was attenuated or contribution from the tributaries was lower than computed from the flood frequency discharges?
- Could localized scour before/during the peak and subsequent deposition during the receding limb of the hydrograph account for lower peak stages than expected? If so, the use of the low roughness value is probably justified because of the limitations imposed by a fixed bed model. However, it also may indicate the need to vary roughness values by flow or depth.
- What is the uncertainty of the surveyed high water marks?
- What values were used for roughness in previous Cowlitz River models? Do past models support a significantly smoother roughness in the downstream reach?

Assuming these questions have been considered and ruled out, explanations should be included in the report due to the low roughness value used in calibration.

NWP Response:

Justification for the roughness values used in the calibrated hydraulic model of the Cowlitz River was presented in the LOP report in terms of published research obtained from a standard reference, the ASCE sedimentation manual. A more thorough, internal, investigation was performed which was not presented in the LOP document and is presented here in order to provide some clarification and justification for the roughness values used in the calibrated model. This explanation expands on the description that was provided in the LOP report and is provided in order to provide an overall sense of the methodology that was used to arrive at the calibrated roughness values and the variation in the Manning's roughness values used to determine $S_{natural}$.

At the onset of the 2009 LOP investigation, the approach to determining a suitable Manning's roughness value for the Cowlitz River was divided into three distinct efforts. First, the Manning's roughness was adjusted based on various calibration techniques. Secondly, the range of Manning's roughness values was computed from predictions of the regime and theory regarding velocity profiles. And finally, roughness values from previous LOP studies were used were compared to current roughness estimates to provide a sense of the overall temporal variation in roughness. Each method is addressed individually in the following paragraphs. Ultimately, the Manning's roughness values selected for the Cowlitz River hydraulic model were intended for use with the LOP study only, where the main concern is estimating rating curves that are to be used in evaluating the probability of levee failure where fragility curves only vary at high stages and high flows.

Regime change in terms of the 2009 LOP analysis is a characteristic that is used to describe changes to the bedform in a sand bed channel, where ripples, dunes, washed out dunes, and antidunes are possible. It is not, however, appropriate to describe a gravel bed channel in terms of the same type of bedforms which are present in the sand bed channel. In the case of the lower Cowlitz, there is a fining trend that occurs from the mouth of the Toutle River, where a significant gravel component is present, to the Columbia River where the channel consists of mainly sand. Indeed, a clear distinction between upper and lower end of the Cowlitz River can be made at roughly RM 10. This distinction is based on changes in channel planform, bed material size, and channel slope. Figure F. 1 shows a plot of D50 over time with respect to river mile for the Lower Cowlitz River.

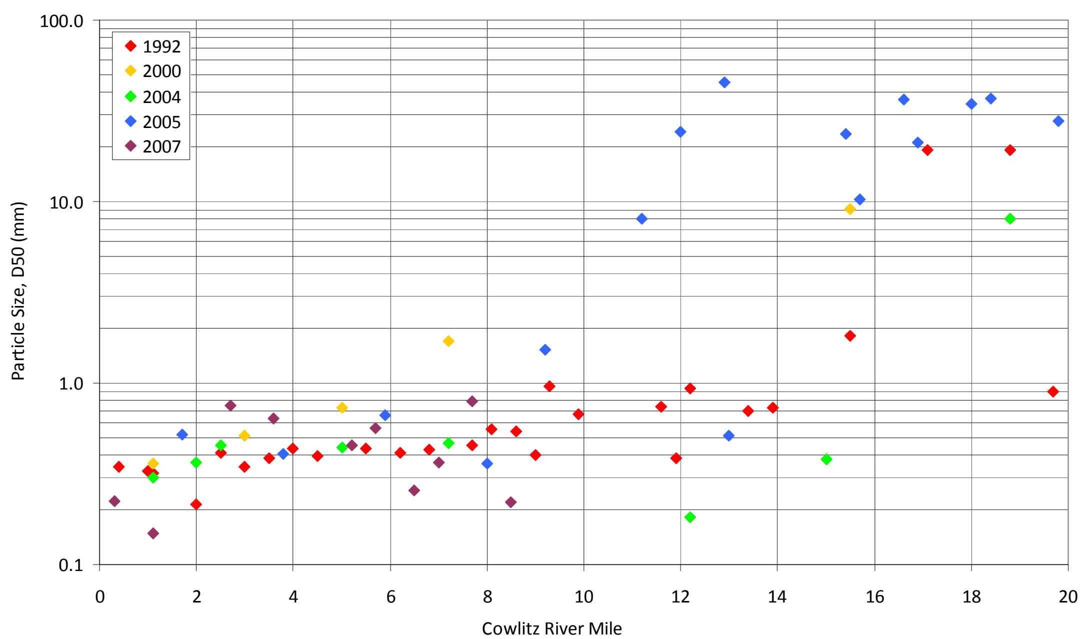


Figure F. 1: D50 Particle Size along the Cowlitz River

As can be seen in Figure F. 1, a discontinuity can be roughly seen at about river mile 10 where the D50 increase substantially. Field observations confirm this discontinuity and as a result, for the purposes of describing the Cowlitz system in the LOP analysis, RM 10 is used as a pivot point below which bed forms that are characteristic of sand bed channels are possible and above which gravel bed mechanics are more appropriate.

As discussed in the LOP report, an effort was made to predict the bedform associated with the Cowlitz River using the Van Rijn's bed form prediction method (Julien, 1998). Results from this method indicate that the bedform reaches transition to upper regime around the 10 percent AEP (~70,000 cfs) below RM 7. For more frequent events the bedform is generally in lower regime dunes or plane bed. However, the probability of levee failure below 10 percent AEP is zero and varying the roughness by discharge will not effect the overall LOP estimate. The focus was therefore centered on refining the estimate of the roughness coefficient for the larger flood events. Upper regime bedform classification was corroborated with a second bedform predictor that relates depth with Froude number. Figure F. 2 shows the relationship between the Froude number and depth developed from a large number of laboratory measurements (Julien, 1998).

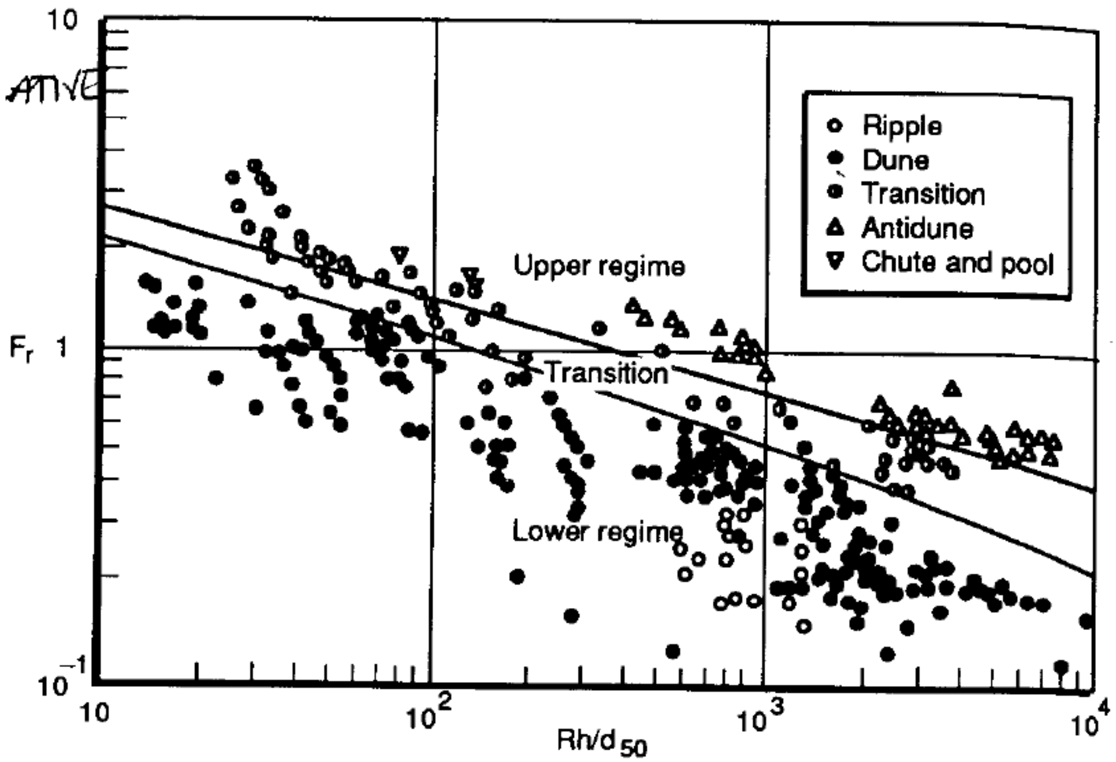


Figure F. 2: Lower and upper regime bedform classification (Athaullah, 1968)

For the lower Cowlitz River the ratio of the radius of curvature to d50 is above 10^3. Froude numbers for the low flows generally fall below 0.3 where Figure F. 2 indicates lower regime and for higher flows the Froude number is approximately 0.4 and above, indicating upper regime.

From theoretical formulations of the velocity profile, an estimate of the Manning's roughness was made based on the d50 present in the channel. The estimates of Manning's roughness show that as the bedform changes from dunes (lower regime) to upper regime (washed out dunes) the roughness value drops precipitously. This result corresponds to empirical data shown in the plot of Manning's n versus discharge in Figure 4-1 of the LOP report. Below RM 10, for the flood with low AEP where washed out dunes are present, computed roughness values were generally in the range of 0.014 to 0.020. Published tables from Julien (1998), which relate Manning's coefficient to bedform, supports the contention that near the upper regime conditions where washed out dunes are present; the roughness coefficient can drop to between 0.014 to 0.020.

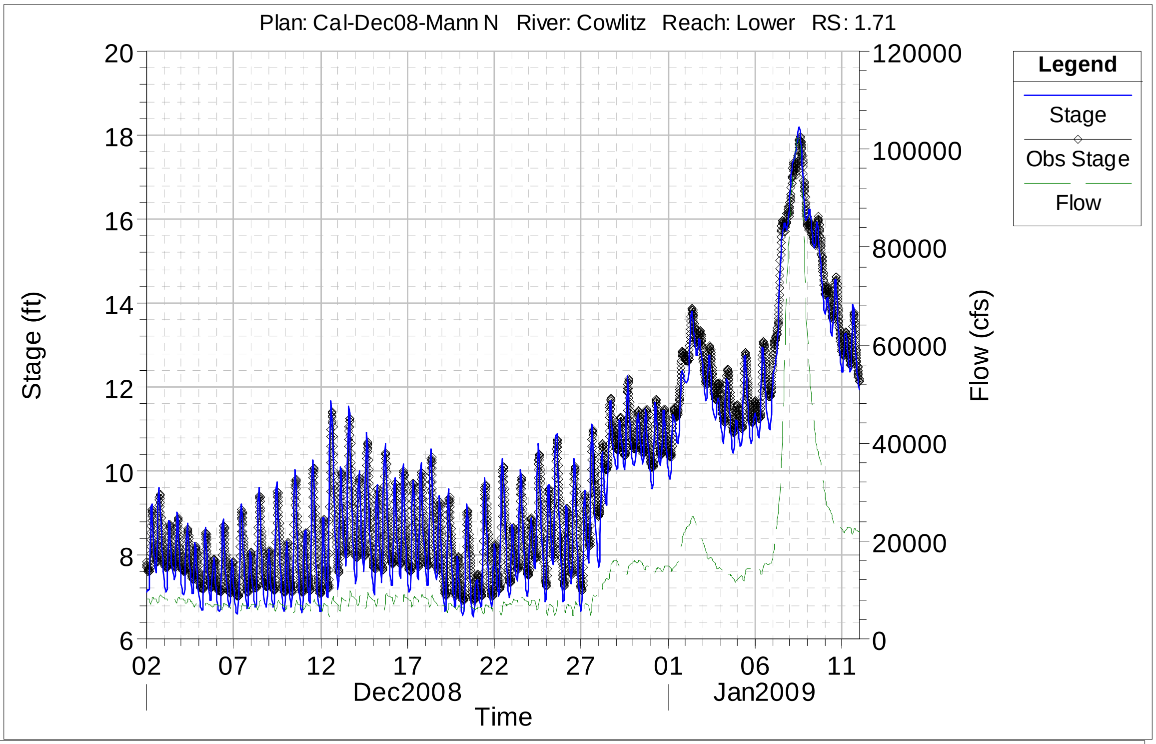
Table F. 1: Typical bedform characteristics (Julien, 1998)

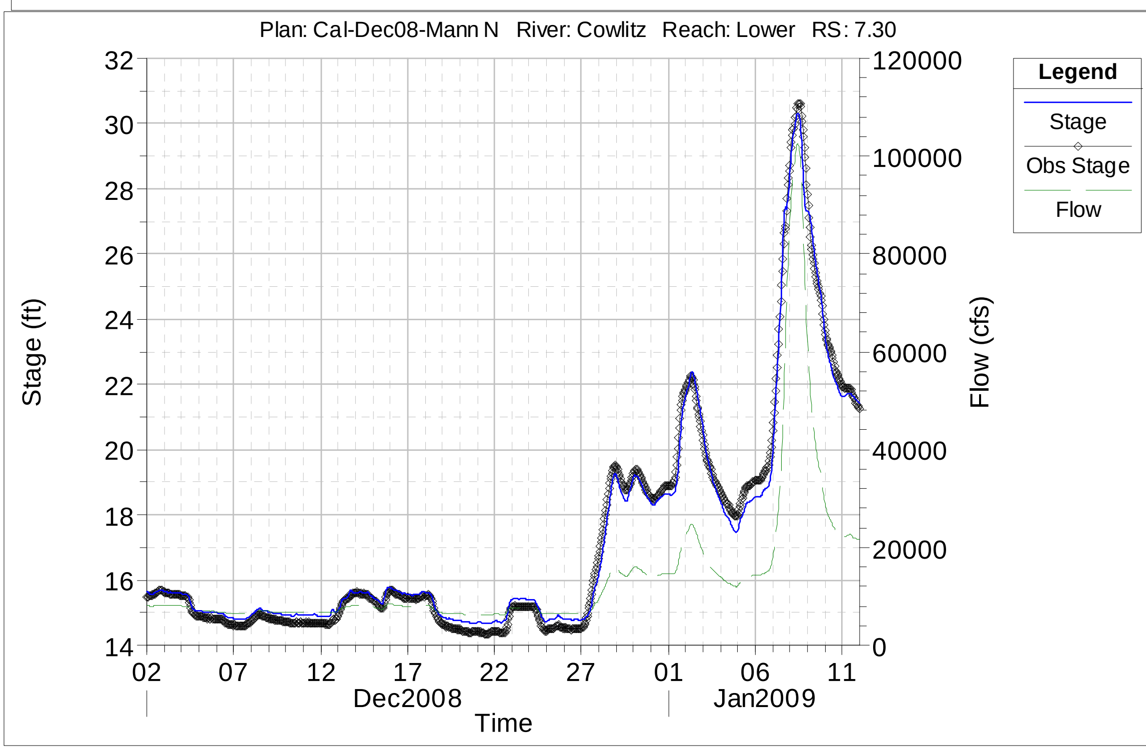
Bedform	Manning coefficient *n*	Concentration (mg/l)	Dominant type of roughness	Bedform surface profiles
Lower flow regime				
Plane bed	0.014	0	Grain	—
Ripples	0.018–0.028	10–200	Form	—
Dunes	0.020–0.040	200–3,000	Form	Out of phase
Washed-out dunes	0.014–0.025	1,000–4,000	Variable	Out of phase
Upper flow regime				
Plane bed	0.010–0.013	2,000–4,000	Grain	—
Antidunes	0.010–0.020	2,000–5,000	Grain	In phase
Chutes and pools	0.018–0.035	5,000–50,000	Variable	In phase

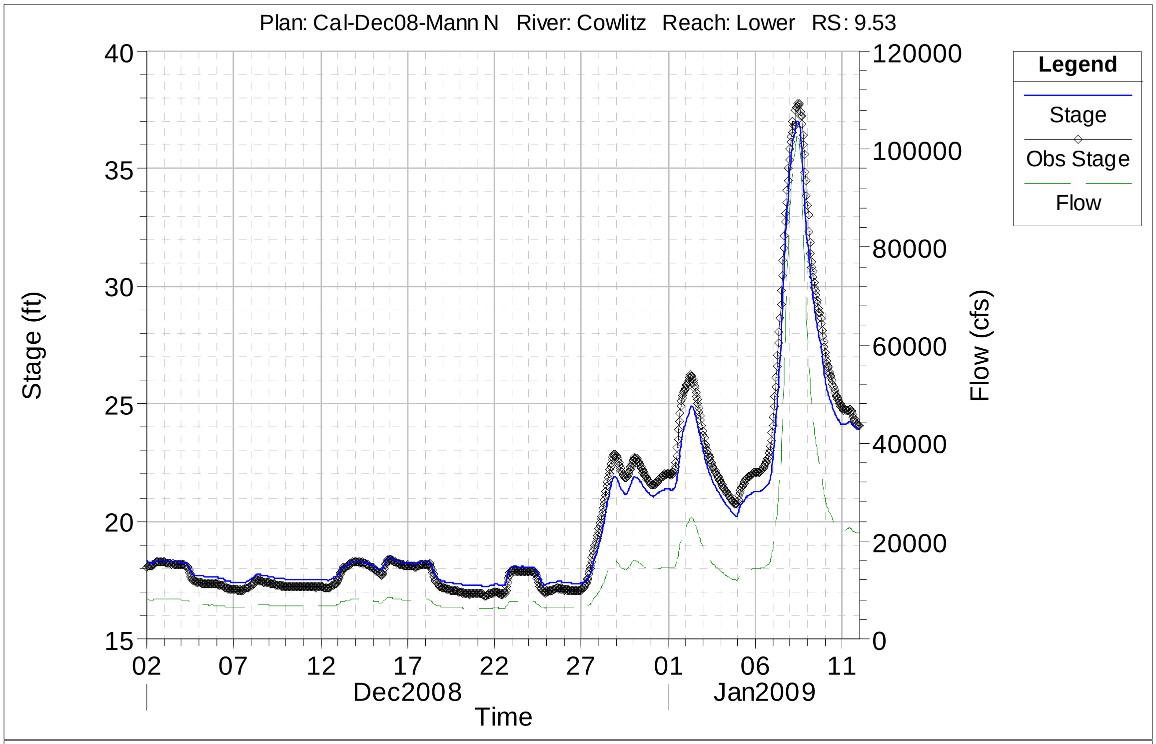
With the range of Manning's roughness determined from research and theoretical computations in mind, two types of calibration were conducted for the lower Cowlitz River. Unsteady calibration to recorded gage data and surveyed high water marks was performed using a Manning's roughness value that varies with respect to discharge and steady flow calibration was also performed using gaged data and surveyed high water marks with a constant Manning's roughness with discharge.

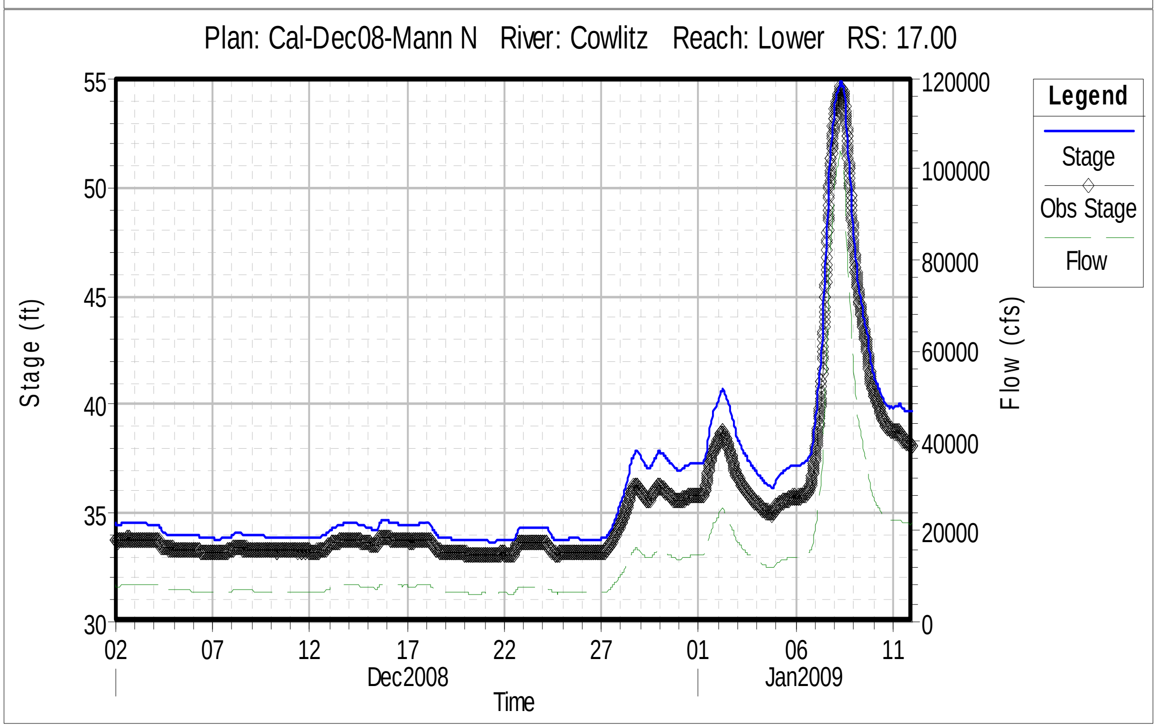
Unsteady calibration

Data from five gages along the Cowlitz were used to calibrate the unsteady flow model for the January 2009 flood event. From RM 0.01 to RM 7.30 Manning's roughness was adjusted with discharge in order to calibrate the stage for the low flows. Roughness values for the peak discharges from RM 0.01 to RM 7.3 are set to 0.02 in the unsteady model. For low flow conditions, leading limb of the flood hydrograph, the roughness values are increased up to a factor of 1.7, or 0.034, in order to match the low flow measured stage hydrograph. Upstream of RM 7.3 to RM 8.07 a roughness value of 0.02 is used, with no variation with flow.


Plan: Cal-Dec08-Mann N River: Cowlitz Reach: Lower RS: 1.71
Stage (ft)
Flow (cfs)
Time
02 07 12 17 22 27 01 06 11
Dec2008
Jan2009
Legend
Stage
Obs Stage
Flow


Plan: Cal-Dec08-Mann N River: Cowlitz Reach: Lower RS: 7.30
Stage (ft)
Flow (cfs)
Time
02 07 12 17 22 27 01 06 11
Dec2008
Jan2009
Legend
Stage
Obs Stage
Flow


Plan: Cal-Dec08-Mann N River: Cowlitz Reach: Lower RS: 9.53
Legend
Stage
Obs Stage
Flow
Stage (ft)
Flow (cfs)
40
35
30
25
20
15
120000
100000
80000
60000
40000
20000
0
02
07
12
17
22
27
01
06
11
Dec2008
Jan2009
Time


Plan: Cal-Dec08-Mann N River: Cowlitz Reach: Lower RS: 17.00
Legend
Stage
Obs Stage
Flow
Stage (ft)
Flow (cfs)
55
50
45
40
35
30
120000
100000
80000
60000
40000
20000
0
02
07
12
17
22
27
01
06
11
Dec2008
Jan2009
Time

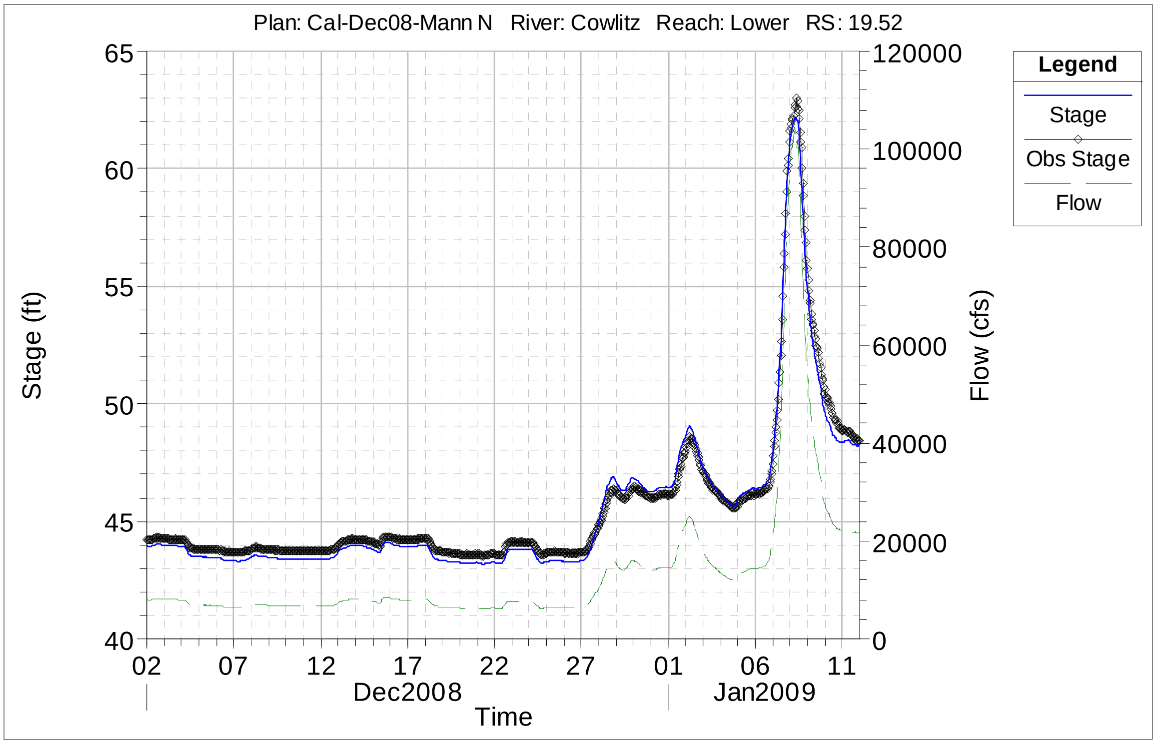


Figure F. 3: Calibration Stage and Flow Hydrographs for the Unsteady Cowlitz River Model

As seen in Figure F. 3, a reasonable calibration was achieved for the unsteady flow modeling. The calibrated roughness values from the unsteady flow model, however, represent a different hydraulic condition than the steady flow model. The unsteady flow model includes, as part of the unsteady computation algorithm, attenuation of the Castle Rock discharge hydrograph. In addition, the unsteady flow model does not include any influence from tributary flow. Since the steady state calibration does include tributary flow and does not include any attenuation, the calibrated Manning's roughness values from the unsteady flow model is not appropriate for the steady state LOP hydraulic model. Furthermore, since zero probability of failure of the levees exists at low flow, variation of roughness values at low flow will not influence the outcome of the LOP analysis. Table F. 2 summarizes the amount of attenuation realized in the unsteady flow model.

Table F. 2: Attenuation realized in the Unsteady flow model

Model	Cowlitz River Peak Flow (cfs)			
	at Castle Rock	below Arkansas Cr	below Ostrander Cr	below Coweeman River
	2008: RM 20.06	2008: RM 16.1	2008: RM 8.64	2008: RM 1.61
Unsteady Flow Discharge	104,000	104,258	102,490	102,236
Steady Flow	106,000	108,925	110,475	118,975
Difference (Steady – Unsteady)	2,000	4,667	7,985	16,739

Steady state calibration was achieved through the use of eight surveyed high water marks and the peak stage from three gage locations. Since the discharge for steady state conditions include tributary flow and does not include any attenuation from Castle Rock to the Columbia River, lower overall roughness values, especially in the downstream reach, need to be used in order to match the observed data. The calibrated roughness values used in the lower end of the model compare well with the computed roughness values for the washed out dune conditions, and the published data in **Error! Reference source not found.**. Essentially, the roughness values that were used to calibrate the computed water surface profile in the steady state model were verified with gage data, measured high water marks, and computed theoretical roughness values. For this reason model variation from Table 5-2 of EM 1110-2-1619 is based on a *good* estimate of Manning's n reliability.

In the past LOP reports, calibration in the lower portion of the Cowlitz River have been hampered by downstream boundary effect. In 1997 the lower seven miles of the Cowlitz River was calibrated to two high water marks at RM 6.3 and RM 7.2 that were surveyed after a flood event that had a significant backwater influence from the Columbia. While comparisons between the 2009 and 1997 calibration profiles were made, it was recognized that the best available data in 1997 was limited and included some erroneous influences from the Columbia River. While calibration from previous studies have provided a limited basis of comparison for the downstream reach of the Cowlitz River, upstream reaches, where ample calibration data have historically been available, reasonable comparisons can be made with the 2009 calibrated roughness values. Upstream of RM 10, the calibrated Manning's roughness values in 2009 compare reasonably well with previously calibrated values from past LOP reports.

In short calibration to the unsteady gage data represents an upper bound of roughness due to the different conditions present in the steady state conditions. Basic assumptions regarding attenuation and peak flooding renders the roughness coefficients obtained from unsteady calibration invalid for steady state conditions. Therefore, an additional calibration effort was performed to develop calibrated roughness coefficients from steady state conditions and input into FDA. Even though a single peak was considered, the calibration event was a significant flood event and provides a reasonable estimate of roughness values needed to model peak events that could ultimately cause levee failure. The highly dynamic nature of the reach (2.7 Mcy of deposition between 2006 and 2008) make calibration to older event less certain and potentially inappropriate. Roughness values at lower flow events might be perhaps different, however, since the probability of levee failure at these high frequency events is zero the variation in roughness is not essential for the LOP analysis and does not effect the estimate. Low roughness values used in the lower portions of the Cowlitz River were verified with theoretical computations, empirical data, published guidance, and observed water surface. To move away from the observed roughness given the quality of the data and the abundance of theoretical support would not be recommended.

<u>NWO Backcheck:</u> **The NWP response is an excellent presentation of the data used in calibration. It is recommended that this data and explanation be either included in the report or at least referred to. The description of the roughness value being factored by up to 1.7 in the unsteady calibration gives more assurance that Figure 4.1 of the report is applicable to the Cowlitz River.**

A table showing the computed n values from both steady and unsteady modeling that clearly illustrates the differences in the two approaches would allow one to feel more comfortable with the low calibrated roughness values.

Per phone conversation with Chris Nygaard on 10Dec2009, it is understood that besides the Castle Rock gage, all other gages are stage gages only. This should be clearly stated to emphasize the need to use the flow frequency curves to determine discharges in the lower reach of the model (rather than measured discharges).

COMMENT CLOSED

2. Consider conditions for the calibration data event. Check for seasonal variation, ice affected stage, and etc.

NWO Comment: OK

3. For historical data, verify that the geometry is similar. Check for any changed conditions (channel degradation or aggradation, bridge construction, levee construction).

NWO Comment: N/A

4. Perform calibration runs combined with separate verification runs if possible.

NWO Comment: No verification runs were made.
NWP Response: Please see response to Model Calibration\Verification comment 1.
NWO Backcheck: **COMMENT CLOSED.**

5. Perform a sensitivity analysis. Items to consider include roughness, expansion and contraction coefficients, boundary conditions, etc.

NWO Comment: Sensitivity analyses were conducted on the roughness and C/E coefficients. The model shows moderate to high sensitivity to roughness and C/E coefficients.
NWP Response: Agreed, this is a relatively sensitive model due to the confined channel and the relative steepness resulting is high velocities.

6. Check boundary conditions for impact on results. Try starting at critical depth, normal depth slope, and a set elevation.

NWO Comment: Boundary conditions used appear reasonable.

Computed Results.

1. Check Q for variation between left, right, and channel. Channel should have the bulk of the flow. Check for flow switch between overbanks.

NWO Comment: OK

2. Check for abrupt changes in top width. Top width should increase as flow event increases (10-, 50-, 100-year events).

NWO Comment: OK

3. Check flow velocity for excessive velocities or rapid change in velocity between sections. Highest velocity is usually in the channel.

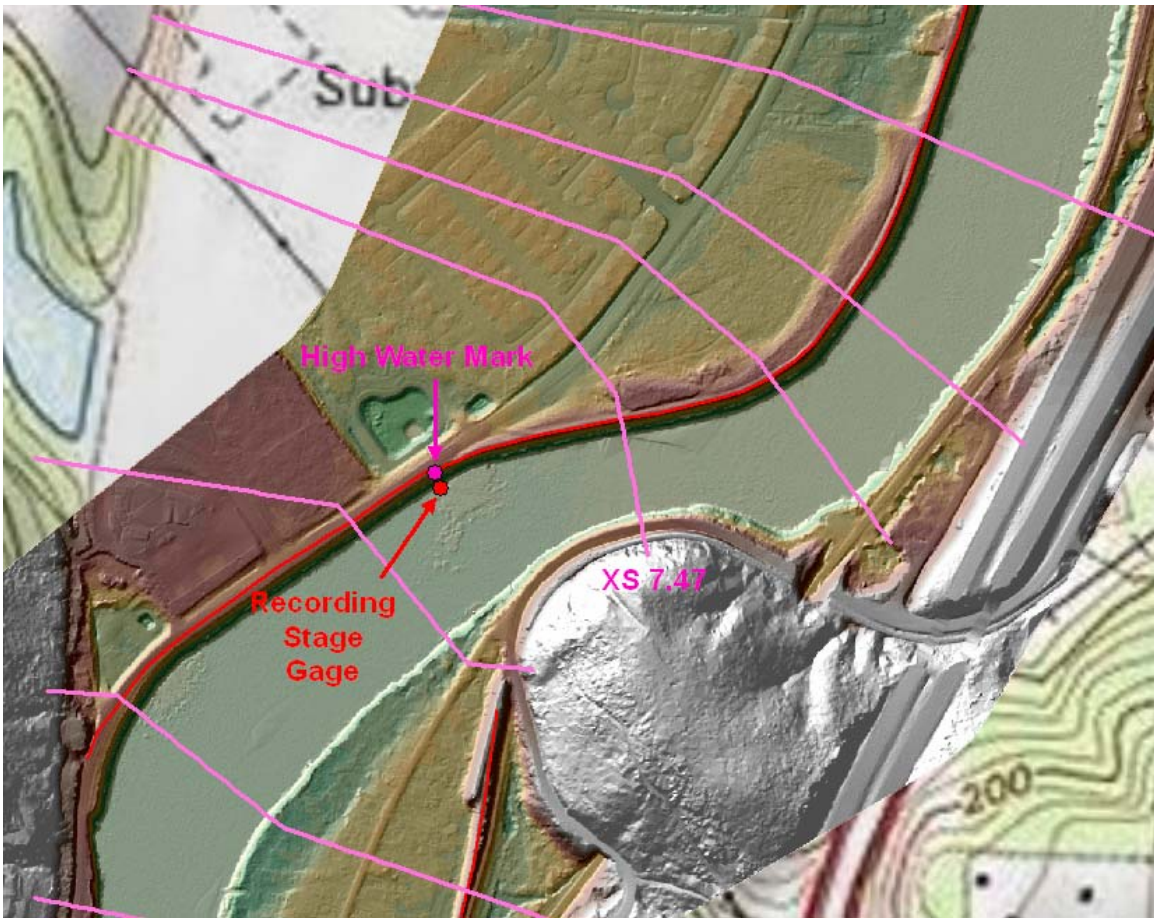
NWO Comment: There are a number of cross sections (≈20) where the upstream computed water surface is lower than the downstream. At these locations, the computed velocity is typically higher than surrounding sections. Typically profile dips are an indication that more or better geometry data is needed. The geometry data used in the analysis appears to be of good quality and obtaining additional surveys is likely cost and time prohibitive. Some explanation of the reason for profile dips should be included in the report.

Verify the channel velocity of >16 ft/s at RM7.47 for the calibration event.

NWP Response: Energy grade lines for all runs look reasonable and very smooth. The high channel velocities modeled during large events (approx 10 ft/sec) make the model very sensitive to contractions and expansions. Variations in the water surface elevation reflect changes in velocity head primarily due to expansions and contractions in the reach. This may result in raises in water surface in the downstream direction given the large amount of velocity head available.

The model consists of high quality bathymetric and overbank data collected very frequently, every 2 to 5 bank widths. Downstream raises in the water surface are seen even in the densest data areas. Language will be added to the report.

The high velocity at RM7.47 is the result of a constriction between a natural feature and a levee. As noted by the high velocity, it is the largest constriction on the reach. A recording stage gage is located at this site and a high water mark collected for the January 2009 event verifies the stage. The model velocity, while very high, is justified given the quality of the input flow, cross sections and verified stages.



NWO Backcheck: **Given the quality and density of geometry data, little can be done to eliminate the profile dips from the model. Because none of the index stations are at a dip location, the impact to the LOP analysis is likely minimal or none. With language added to the report, COMMENT CLOSED**

4. Check the difference in water surface and energy grade elevation between sections to find high loss sections and verify.

NWO Comment: OK

5. Check the Froude Number and the occurrence of critical depth. Check a mixed flow analysis. If supercritical flow is indicated, check roughness and geometry. Compare with site visit observations, try to verify that supercritical flow can actually occur.

NWO Comment: OK

Review of Hydraulic Uncertainty

PURPOSE
This document contains review comments pertaining to the hydraulic uncertainty methodology and computations for the Cowlitz River Level of Protection update (Portland District).

INTRODUCTION
The following comments are based on a review of the HEC-RAS model and uncertainty computations from Chapter 4: Hydraulic Analysis, Appendix C: Cowlitz River Hydraulic Model, and Appendix F: Uncertainty Results of the "Cowlitz River Levee Systems, 2009 Level of Flood Protection Update Summary" (USACE Portland District, 2009). The comments are numbered to facilitate responses and backchecks, but are not in any particular order.

COMMENTS

1. Overall, the computations to determine the hydraulic uncertainty appear reasonable and the methodology used to combine the various sources of uncertainty is sound.

<u>NWP Response:</u> Agreed.

2. Manning's roughness variations used to determine $S_{natural}$. Considering the variability in the calibrated roughness value over the 20 mile reach (n=0.015 to 0.035), a slightly larger deviation from the calibrated roughness value for the sensitivity analysis should be considered. Figure 5-4 of EM 1110-2-1619 indicates that for n=0.035, the standard deviation of n value estimate would be closer to 0.009, rather than 14% variation (±0.004-0.005) used in the sensitivity analysis. While Figure 5-4 is only intended to be a rough guide, it would seem that if calibrated roughness values over 20 miles vary so widely, the roughness variation for the sensitivity analysis might be larger.

   If a more intense evaluation was completed to justify only varying the roughness by 14% based on the cited references, it should be documented in the report.

   Varying the roughness values for the lower reach by 30% seems reasonable considering the low value used in calibration.

<u>NWP Response:</u>
The upper 10 miles of the study reach are more easily and historically more frequently calibrated due to two reasons. The lower portion is both tidal and affected by backwater from the Columbia River while the upper portion is standard open channel flow. The upper portion also contains a long term USGS stage gage at RM 17 facilitating calibration the reach immediately below the gage. Previous LOP reports have indicated that the upper portion of the reach could be calibrated with high water mark and gage date while the lower portion was left to engineering judgment due to a lack of recording gage data and uncertainty on whether the high water marks were created by tidal backwater effects or Cowlitz inflow. Mannings roughness through three available calibrations (1996, 2006 and 2009) vary between 0.025 and 0.035. This range is expressed more accurately with the reported variation than that provided by guidance. As guidance is this case is a supplement to other data, we defer the historic variation in the upper section which has reliably had high quality calibration data.
<u>NWO Backcheck:</u> **COMMENT CLOSED**

3. Uncertainty due to model and data limitations (S_{model}). Based on the calibration data presented in the report, the model was only adjusted to one set of high water marks. In order for the reliability of the roughness value to be considered "Good", the model must be adjusted <u>and</u> validated to a stream gauge, a set of high water marks in the project effective size range, AND other data (Table 4-2). If other calibration data was available and used to adjust the model, the supporting data should be presented in

the report. If no other calibration/validation was accomplished, consider changing the Manning's n value reliability to fair to poor.

NWP Response: The roughness values that were used to calibrate the computed water surface profile in the steady state model were verified with gage data, measured high water marks, and computed theoretical roughness values. For this reason model variation from Table 5-2 of EM 1110-2-1619 is based on a *good* estimate of Manning's n reliability. Please see response to Model Calibration\Verification comment 1 for further explanation.

NWO Backcheck: **Based on the additional data presented in the response to comment 1, Model Calibration/Verification, this comment is resolved. COMMENT CLOSED**

4. Computations and justification used to determine the uncertainty due to sedimentation appear sound. The evaluation of the variation in roughness due to regime change may be more appropriately considered in the uncertainty due to natural variations as it does not provide an indication of overall erosion/deposition or degradation/aggradation trends.

NWP Response:

Sedimentation uncertainty as related to level of protection updating is to determine the uncertainty of the calculation due to sedimentation and sedimentation processes. Variation in roughness due to regime change was viewed as a dynamic sedimentation process that had the ability to affect the water surface elevation.

Investigation into variation of roughness due to bed form change in the lower 10 miles where the stream is sand bed and capable of generating bed form indicates that dunes begin to wash out on average near 70kcfs or the 20 AEP event. It is calculated that roughness drops quickly once dunes begin to wash out. Expected stages do not approach safe water elevation until roughness has theoretically, and through observation in January 2009, dropped to low levels in the lower 9 miles. Therefore, the variation due to bed form that affects level of protection is the variation in bed form when the dunes are washing out and transitioning to antidune and very low. Including a large variation in roughness at levels below the 0% chance of failure in the fragility curves, as is done in the analysis, is conservative in terms of uncertainty.

NWO Backcheck: **COMMENT CLOSED**

F.4. Level of Protection Review

The level of protection review was performed by Mike Deering, Beth Faber, and Woody Fields of HEC. Comments were received on December 22, 2009 in the form of a spreadsheet. These comments and the corresponding responses are included below.

Reviewer	Page	Section	Reviewer's Comment	Responses	Backcheck
Fields	30	5.2	Table 5-3 has not been updated to show the correct low LOP values for each levee.	The Table has been updated.	Great. Comment Closed
Fields			The fragility curve in the FDA model has been incorrectly input for Lexington index point 1 (LXIP1). By updating the fragility curve to a linear interpolation between the safe water level and top of levee the values change for that reach considerably. 250 year event changes from 88.5% to 77.8% and the 100 year event changes from 97.8% to 95.5%	Fragility curve at this location was defined based on the Safe Water Survey. Curve checked and is inputted correctly. See addendum in Appendix D, the first table.	Concur. Comment closed.
Fields		Hydraulics	It should be stated why the RAS model was only calibrated to a single high flow event. Was the gage data at lower flows unreliable? Uncertainty should be increased when only calibrating to a single flow event. I believe that the hydraulics reviewer mentioned this but I did not read any reasoning or explanation in the text.	The calibration effort for the hydraulic model was tailored for the LOP estimate specifically. For the LOP, the focus is on the high flows primarily concerned with the frequency events near failure. Calibration of low flow conditions, or varying roughness values based on discharge, was considered inappropriate for this application.	Concur. Comment closed.
Fields		Appendix F	Longview index point 3 (LVIP3) stage discharge curve discharge 11,700 flow should have a standard deviation of 0.781 according to the document. The FDA model has a standard deviation of 0.871. Changing the value in FDA to 0.781 does not change the outcome.	The incorrectly entered value was changed to the correct value.	Good. Comment closed.
Fields	13-23	4.3	The lengthy hydraulic uncertianty explanation given in section 4.3 should	The explanation of sediment uncertainty in Section 4.3 has been moved to the Appendix	Good. Comment closed.

			be placed in Appendix C and the uncertainty explained in the document text should be short like the hydrologic and geotech sections 3.4 and 2	C.	
Fields		Appendix F	Appendix F could/should be included in Appendix C since it is related to the hydraulic modeling. Also, the tables in Appendix F are mislabeled for appendix E. Again, the lengthy text on hydraulic uncertainty should be placed in this appendix.	The summary of uncertainty included in Appendix F has been moved to Appendix C.	Good. Comment closed.
Faber	4	1.4	Last sentence of hydraulic risk paragraph, suggest "....ranging from the 99.99% to 0.01% exceedance probability in any given year."	This change has been made.	Good. Comment closed.
Faber		2	The levee analysis doesn't seem to meet the same uncertainty analysis standard as the other compontents of the LOP analysis.	The levee fragility curves are the result of a detailed (approx. 100 pages) levee study and a simplified approach in which the Pf = 0 at the safe water level (where a combination of analyses and past performance indicate negligible Pf) and the Pf = 1 at the levee top (due to high certainty of failure due to overtopping of the predominantly non-cohesive levees), and a simple straight-line assumption between these two points. The levee study divides the levees into reaches of similar characteristics and considers sources of uncertainty, such as levee performance under various potential failure modes (seepage, stability, riverside scour) and the track-record and capabilities of the sponsor during flood events, in defining the safe water level. Index points were established at potential "weak" points and other points of interest along the levee for calculation of level of protection. For Lexington and Coweeman levees, where underseepage is critical potential failure mode, a pseudo-reliability approach (described in the Safe Water Level Study Addendum) was used	The comment was on the uncertainty analysis, not on the physical levee analysis. Perhaps in the future, a less simplified approach can be used. For now, comment closed.

				because the straight-line approach was considered too conservative.	
Faber	9	3.2	Should mention the historical flood data considered, along with mention of the systematic regulated and unregulated records.	Text has been added to Section 3.2 that describes the study of the historical data.	Good. Comment closed.
Faber	9	3.2	Suggest change "hypothetic" storm patterns to "hypothetical."	Updated	Good. Comment closed.
Faber	11	3.5	Suggest referring back to appendix B for more description.	Updated	Good. Comment closed.
Faber	14	4.3.2	The mean or "expected" stage is not by definition the 50% stage. The 50% stage refers to the median. If intending to state that mean and median are assumed the same (ie, symmetrical uncertainty) this should be said explicitly.	Reference to the 50 percent stage has been removed as it wasn't crucial to the context.	Good. Comment closed.
Faber	14	4.3.2	"2 standard deviation confidence interval" would more correctly be called the "4 standard deviation confidence interval."	Updated	Good. Comment closed.
Faber	16	4.3.4.1	Says figure 4-3 suggests uncertainty can vary from 0.6 to 0.8 ft. The figure doesn't clearly suggest this, and perhaps this final estimate comes from more information. Should be stated more specifically.	The range given was change to a more appropriate value of 0.6 to 1.2. Text was added to indicate that this range is based on the range of discharge that is relavant to LOP, or near the 100-year frequency event.	Good. Comment closed.
Faber	24	5.1	Says "Results from FDA are reported in terms of probability of failure at a given index point for a given non-exceedance probability event." This is incorrect. Suggest "Results from FDA are reported in terms of a non-exceedance probability (or assurance) of containing a given %-chance exceedance event.	This change has been made.	Good. Comment closed.
Faber	27	5.2	Table 5-3 doesn't agree with table 5-2	Updated	Good. Comment closed.
Faber/Fields	26	5.2	How was the current level of protection determined from the FDA	Must look at the output from the FDA runs to determine the exceedance that corresponds	Concur. Comment closed.

			output? We cannot reproduce the method.	to the 90 percent probability.	
Deering	ES-1 and Page 3	Bullet 4 and 1.1	Suggest referring to levee failure function as "fragility curve" and not PNP.	Updated	Concur. Comment closed.
Deering	5	1.3	Suggest renaming this section "Authorized Level of Protection" and describe what is meant by this terminology. e.g. Congress authorized this project to contain the X frequency flood with top of levee profile set at the computed water surface of that frequency discharge.OR, Congress authorized this project to contain 118,000 cfs which is the flow of the 19XX flood and includes Y feet of freeboard to assure containment of that discharge. Additionally, delete reference to ETL as new EC will not contain that definition of "level of protection". The EC definition will align more with the frequency of event that is contained with a high level of assurance i.e. 90%. This is extremely important so as not to compare apples and oranges.	In the 1980s and early 1990s, the levels of protection for Castle Rock, Lexington, Longview, and Kelso were determined using a deterministic approach in which median values of flood stages were compared to levee safe water levels (SWL). The SWL was evaluated as the highest flood level for which reasonable assurance could be made that the levee would not fail, and was restricted to no less than 3 ft below the levee top in order to provide freeboard for uncertainties. The SWL was often dictated by encroachments to the levees. The level of protection was evaluated as the highest average-return-period-event whose median-value flood profile was no higher than the SWL at all points along the levee	Thanks for the explanation. Comment closed
Deering	5	1.4	Suggest stating the definition "Level of Protection" has stated in Section 5.2 in terms of CNP or Assurance.	Additional text was added to clarify definition of LOP in Section 1.4.	Concur. Comment closed.
Deering	5	1.4	Item 1 - Suggest using "levee condition OR "levee stabilty" instead of "levee risk" as risk implies consequences which is not being included in this analysis.	The term "risk" has been removed from the document.	Concur. Comment closed.
Deering	5	1.4	Item 1 - Suggest rewording first sentence as "Geotechnical or levee stability as defined by the determination of probability of failure versus stage function (fragility curve)	Updated	Concur. Comment closed.

			at each index location.		
Deering	5	1.4	Item 1 - Delete reference to ETL.	Updated	Concur. Comment closed.
Deering	5	1.4	Item 1 - Suggest rewording "levee risk" with "levee condition".	The term "risk" has been removed from the document.	Concur. Comment closed.
Deering	5	1.4	Item 2 - Suggest using "hydrologic condition" instead of "hydrologic risk" as risk implies consequences which is not being included in this analysis.	The term "risk" has been removed from the document.	Concur. Comment closed.
Deering	5	1.4	Item 3 - Suggest using "hydraulic condition" instead of "hydraulic risk" as risk implies consequences which is not being included in this analysis.	The term "risk" has been removed from the document.	Concur. Comment closed.
Deering	5	1.4	Last Sentence - Suggest rewording "three risk factors" to "three key factors".	Updated	Concur. Comment closed.
Deering	27 and 30	5.2	Table 5-2 and Table 5-3 - Care should be taken in comparing Authorized LOP and Current LOP columns depending on what the definition of Authorized LOP is.	The authorized LOP includes uncertainty in the form of freeboard. Although it is recognized that current guidance supports a different methodology the basic components that allows for comparison are present. Also, the current method used to determine LOP follows Corps guidance and has historical precedence given multiple years of previous LOP reporting beginning in 1997. A description of the origin of the authorized LOP is included in the Introduction.	Concur. Comment closed.